"LANDSLIDE INVESTIGATIONS, MAIN NORTH LINE RAILWAY, SOUTH ISLAND, NEW ZEALAND"

A thesis
submitted in partial fulfilment
of the requirements for the degree
of
Master of Science in Engineering Geology
in the
University of Canterbury

by

G. Guy Grocott, BSc, BSc (Hons.)

with 3 separate charts

University of Canterbury
May, 1977

Frontispiece. The effect of slope instability on communication routes: storm damage following the passage of Cyclone Alison over the steep coastal Kaikoura district, South Island of New Zealand (March, 1975).

CONTENTS

PAGE

PAGE

PAGE

LIST OF TABLES

TABLE

LIST OF FIGURES

FIGURE

LIST OF PLATES

The following notation is used in this study:

m	:	metre
m^2	:	metre squared
SG	:	specific gravity
g	:	acceleration due to gravity
p	:	density, in general
p_d	:	dry density
p_s	:	saturated density
Ip	:	plasticity index
Id	:	slake durability index
k	:	coefficient of permeability
S	:	resistance to shearing
6	:	total normal stress
$6'$	:	effective normal stress
$6'_1$	:	major effective stress
$6'_3$	:	minor effective stress (cell pressure)
$(6'_1-6'_3)$	:	deviator stress
C	:	cohesion, in general
C_u	:	cohesion (total stress)
C'	:	cohesion (effective stress)
C_r	:	cohesion (residual value)
Ø	:	angle of internal friction, in general
$Ø_u$	:	angle of internal friction (total stress)
$Ø'$	:	angle of internal friction (effective stress)
$Ø_f$	:	angle of internal friction (peak value)
$Ø_r$	:	angle of internal friction (residual value)
W	:	weight of sliding block
U	:	pressure due to pore water on sliding surface
V	:	force due to water pressure in tension crack

A	:	area, base area of sliding surface
ØF	:	slope angle
ØP	:	angle of sliding
H	:	height of slope, or height of slide mass above shear surface
Z	:	height of tension crack
Z_w	:	height of water in tension crack
D	:	distance of tension crack from slope crest
R_n, R_{nh}	:	lateral soil forces, equal, but opposite
F	:	factor of safety

ABSTRACT

The most important section of railway in the South Island of New Zealand, the Main North Line, offers a continuous rail link serving the northern half of the South Island's east coast. The Main North Line, operated by the New Zealand Railways Department, extends 348 kilometres between Christchurch (in the south) and Picton (at the northern tip of the South Island). At some 25 to 30 sections of the line, speed restrictions imposed on rail traffic have reduced Christchurch-Picton travel times by approximately one hour. Many restrictions occur at sections of the line where instability in natural slopes, cuts and fills gives concern to track safety. This study summarises investigations and recommends correction measures for three areas of slope instability through which the railway passes.

The Ethelton Slip, an earth movement in a natural slope, is sited 101 kilometres north of Christchurch. The Main North Line passes for 450m across the toe of the landslide. The site covers an area of 30 hectares, with relief between the upper and lower boundaries approaching 200m. Slope angles over the surface range to 45°. At the turn of this century, movement on the railway reached 0.5-1.0m per week. This study has shown that movement is not occurring generally over the site, but is restricted to a small (one hectare) zone above and including the railway. Displacement of this zone reached 13.1cm over a 9.7 month survey period; numerous shear and tension cracks over much of the zone infer movement is deep seated. Material forming

the slide mass is a colluvium, derived by deep weathering and erosion of a basic volcanic bedrock intrusive into sandstones and mudstones of Triassic age. Slope failure is believed to have resulted from a process of overstressing within the weathered slope mantle as a result of over-steepening and undercutting at the toe of the slope through river action. Surface drainage and slope regrading is recommended to eliminate water infiltration into the slide mass. If movements persist at the site following these measures, horizontal or counterfort drains could be considered as additional controls.

The second area considered in this study, a one kilometre-long railway cutting, causes perennial track clearance and maintenance problems from the numerous mudflow and earthfall-type landslides originating from the batters above both sides of the track. The Hawkswood Cut, located 140 kilometres north of Christchurch, has batters originally rising at 45^{o} and approaching 20m in height. Near the centre of the Hawkswood, instability of the cutting sides has advanced to the stage where slopes are approaching the vertical and remedial attention is a necessity. During high intensity or prolonged rainfall, small earthflows become mobilized from the batters as a result of surface runoff leading to a reduction in the effective normal stress of surficial slope material. Earthfalls, or soil blocks breaking away from the slope faces, are also associated with periods of heavy rainfall. Earthfalls result from seepage pore water pressures exerting on unfavourably orientated, stress-relief-induced fissures. Earthworks to reduce the present slope angles are shortly to be undertaken. A continuous (without benching), near-30^{o} batter is recommended for the new cutting, and surface drainage and slope regrading above the cutting to permit surface runoff is strongly advised.

The Mikonui earthflow forms an extensive area of natural slope movement over a distance of 1.3 kilometres from the Kaikoura coast (east coast, South Island), inland to a point 215m above sea level. The Main North Line crosses the toe of the landslide at the coast. The active width of the earthflow tapers from 200m measured parallel to the coast, to 25-30m at the upper boundary of the site. The lateral boundaries of the earthflow are marked by discreet shear zones. The basal sliding surface is coincident with the upper surface of an insitu, late Cretaceous-age, Ca-rich bentonite immediately underlying the slide mass. The slide mass attains a thickness of over 30m near the lower boundary of the site. Surface survey stations were displaced a maximum 91.1cm over a 10 month survey period during this study. Slope regrading to eliminate ponds, swampy areas, and hummocky topography, and an extension of surface drains already constructed at the site, are recommended to encourage surface runoff. Gravitational driving forces causing movements in the earthflow may be reduced by the excavation of an area of secondary landsliding above the site. The installation of subsurface drainage should further increase slope stability. Elimination of the earthflow by bridging the railway across the toe of the landslide is likely to be the only long-term, permanent control.

SECTION 1: GENERAL

1.1 INTRODUCTION

The elevation to A-grade status in 1971 of New Zealand Railway's Main North Line, running between Christchurch and Picton in the South Island of New Zealand, demonstrates the increasing importance placed on this section of railway as a means of transportation. The rise in status of the line, justified on a freight density (gross tonne-kilometre) basis, is related principally to the occurrence of significant historical events in the national transport industry.

Introduction of the Blenheim-Nelson notional railway (subsidised road transport) system in 1957, coupled with the commencement of Wellington-Picton inter-island rail ferries in 1962, 1967, 1972 and 1974, has allowed a recent growth rate in freight of some 5-11% per annum on the Main North Line. With rail freight transportation costs currently one third that of road, per tonne-kilometre, and the recent cancellation of Wellington-Lyttelton inter-island shipping services in 1976, the importance of the Main North Line should further increase.

Currently 120 plus locomotives per week transport approximately 3 million gross tonnes per year of freight between Christchurch and Picton. A single daily passenger service connecting with the Picton inter-island rail ferry service provides a cheap Wellington-Christchurch link.

The Main North Line has a somewhat tarnished reputation, perhaps erroneously, as a railway plagued by blockages. Reliability figures for 1971-74 reveal the line to have been available for the continuous running of traffic 98.29% of the time. The number of hours lost through blockages for 1971-74 averaged 149 per annum: 67% through derailments, and 33% due to phenomena such as landslips, winds, floods, washouts and ballast failures.

Reliability figures for the Main North Line during 1975, for which the number of hours lost through blockages exceeded 580, demonstrate the vulnerability of communication routes to disruption from natural catastrophes such as floods, landslides and storms.

The passage of a cyclone, Cyclone Alison, over New Zealand on 11-12 March, 1975 caused widespread damage to exposed eastern and northern districts of the country from the strong, wet, east to northeast airflow which accompanied the storm. A narrow, 100 kilometre long strip adjacent to the coastal Kaikoura district (east coast South Island) experienced more than 500mm of rain, and six-hour rainfall intensities exceeded 30mm per hour. Damage to the state highway and Main North Line, which run side-by-side over much of this coastal strip, amounted to NZ$1.75M from landslides, washouts and flood debris. Rail traffic were operational again after 12 days. Slope movements during Cyclone Alison in the steep Kaikoura region were principally restricted to shallow-seated regolith-type slides involving rapid transportation of weathered surficial slope debris.

Both Dj and Dg locomotives operate individually, or in tandem, over the 348 Main North Line kilometres. The 1062mm gauge track is approved for speeds up to 50 kph for freight trains and 80 kph for passenger services.

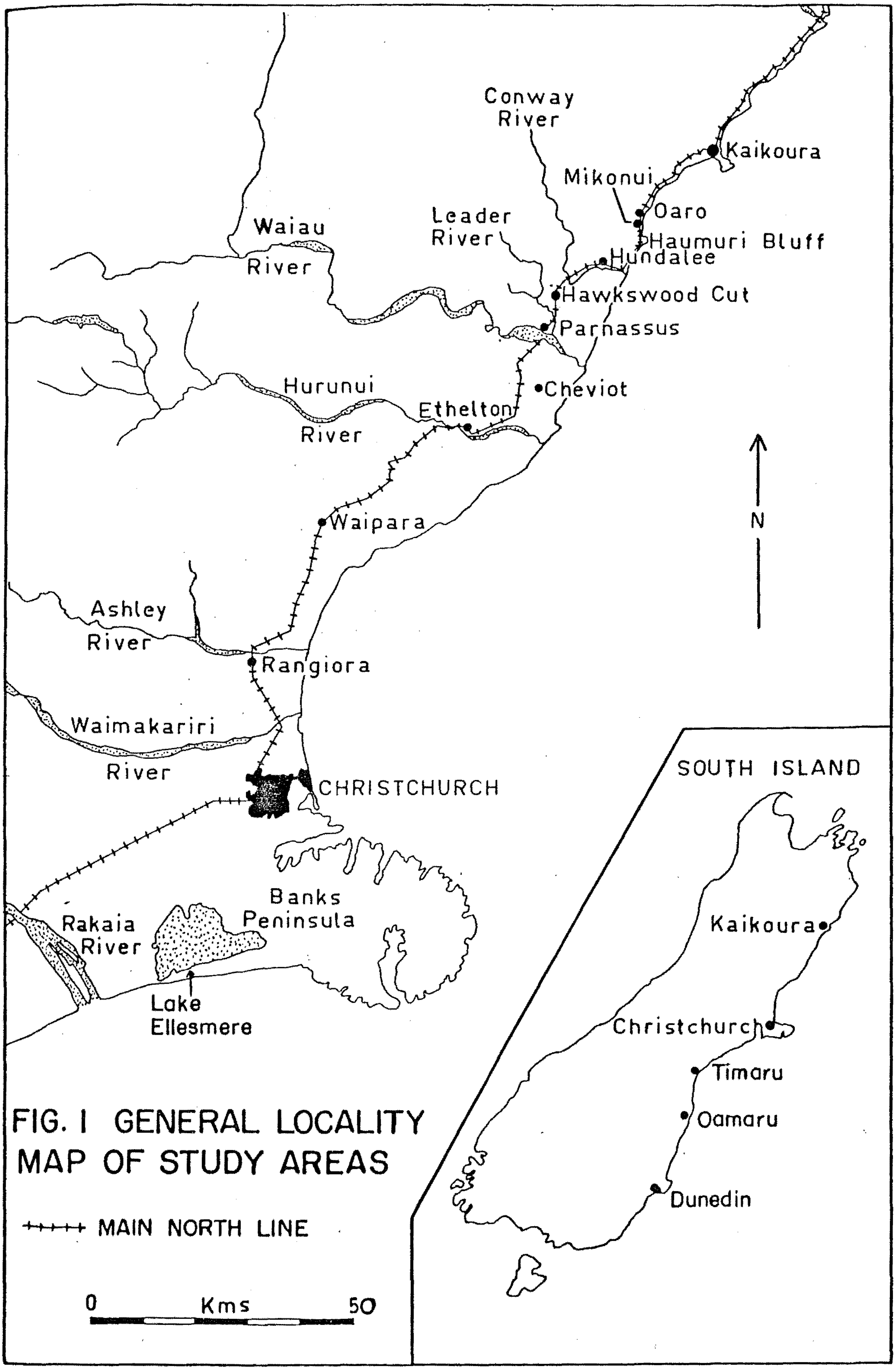


FIG. 1 GENERAL LOCALITY MAP OF STUDY AREAS

At some 25 to 30 sections of line, speed restrictions of 10 or 25 kph are enforced on rail traffic. Many restrictions occur in areas where instability of natural slopes and cuts is giving concern for track safety. A programme of track-improvement, in which widening of cuts and fills and improvement of drainage and track access is presently being undertaken, should reduce the number of restrictions. In addition, increasing the weight of rail to 50kg/m and a comprehensive sleeper relayer programme are also being undertaken.

Improvements to Christchurch-Picton passenger service times up to one hour will probably occur following a reduction in the number of speed restrictions. The beneficial effects of track-improvement are possibly already being experienced; during 1976 only 60 hours were lost through blockages, a considerable reduction on the 1971-74 figures.

1.2 SCOPE OF REPORT

This thesis summarises investigations into three areas of slope instability through which the Picton-Christchurch Main North Line passes. At two of the localities the railway crosses landslides formed in natural slopes, while at the third, batter instability along an excavated cutting results in perennial track clearance and maintenance problems.

A brief history of the problem is outlined for each locality. Landslide descriptions, postulated failure mechanisms, and field investigations undertaken at each site are discussed. A general review of earth movement correction measures is given, and those controls applicable or not applicable to any of the study areas discussed. Finally, the preferred remedies for the three landslides are given as a list of recommendations for each of the study areas. Emphasis is placed on those remedies likely to be most economically acceptable to the New Zealand Railways, even though such measures are likely to effect only a partial control of the slope movements.

Fifteen months were spent on the compilation of the report, of which more than four were spent in the field.

1.3 PHYSIOGRAPHY OF THE STUDY AREAS

1.31 Names and Locations

The study areas in general lie between 80 and 130 kilometres north of Christchurch within the north eastern districts of North Canterbury and the coastal Kaikoura region of southern Marlborough, South Island, New Zealand (Fig. 1).

The three areas to be discussed, with their detailed locations, are named as follows (Figs. 1 & 2):

(a) The Ethelton Slip, the southern-most landslide, is situated 101 kilometres north of Christchurch on the Main North Line* and approximately one kilometre north of the Ethelton Railway Station.

(b) The Hawkswood Cut, located at 140 kilometres, is approximately midway between Parnassus and Ferniehurst Railway Stations.

(c) The Mikonui earthflow, sited at 165 kilometres

* Main North Line distances from Christchurch are taken from a survey control at Addington Station (Fig. 2).

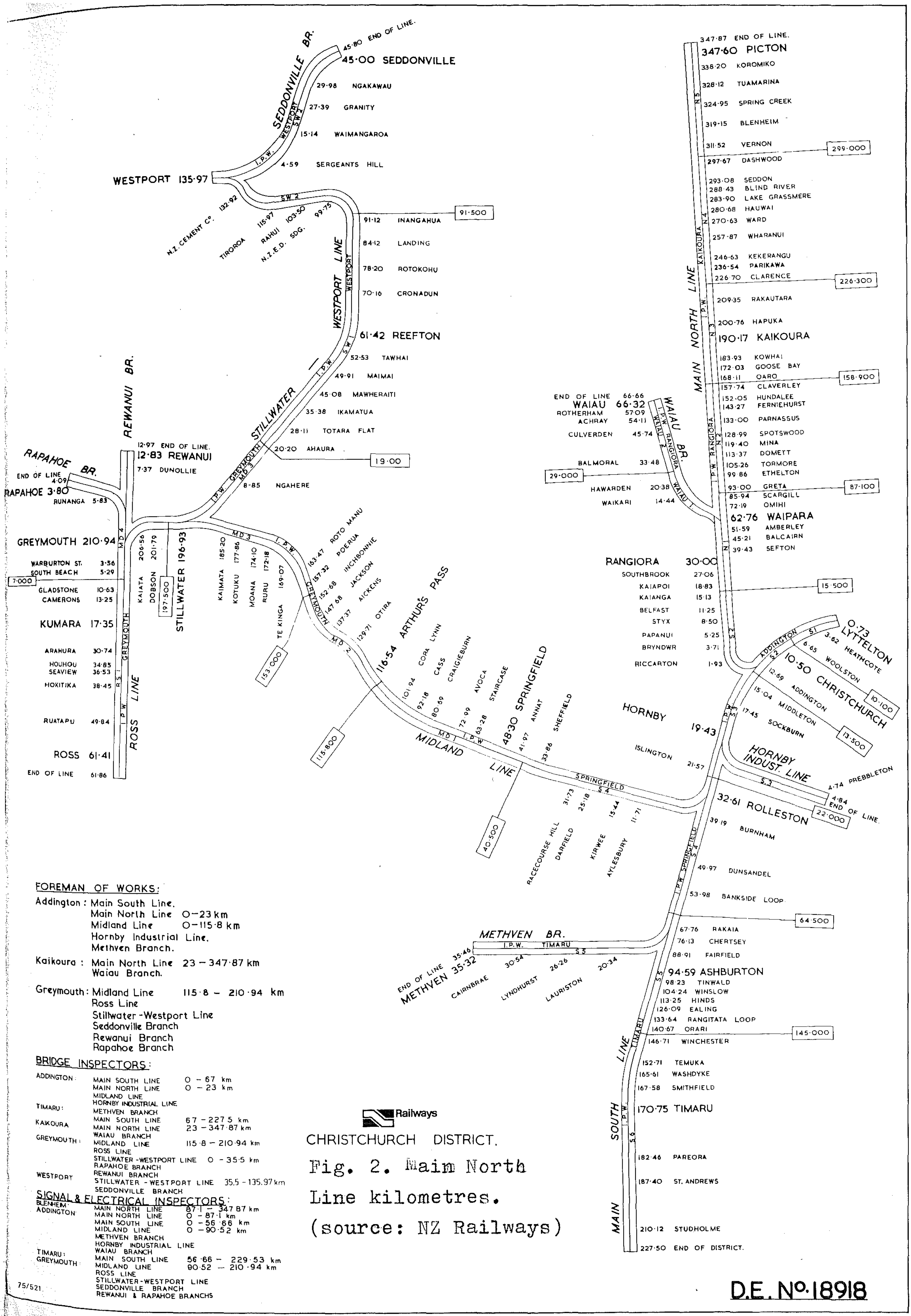


Fig. 2. Main North Line kilometres. (source: NZ Railways)

at a point three kilometres south of Oaro and five kilometres north of Claverly Railway Stations.

1.32 Geomorphology

The largest area of flat land in the South Island, the Canterbury Plains, stretches some 240 kilometres along the eastern coastal strip from approximately the mouth of the Hurunui River (in the north), to approximately Oamaru in the south. The Canterbury Plains comprise an area of gentle, undulating topography of low relief. The plains are an accummulation of substantial thicknesses (10-12,000m) of glacial outwash gravels, deposited as coalescing piedmont fans during glacial periods in New Zealand's recent geological history.

To the northwest and northeast the Canterbury Plains rise into a series of foothills before ascending steeply to the Southern Alps, the largest mountain ranges of New Zealand. These axial ranges traverse the central South Island as an alpine "backbone".

The study areas are located within foothills of the Southern Alps in the north eastern extremity of the Canterbury Plains. Here steep-sided slopes, with moderate relief up to 1000m, formed by rapid downcutting during approximately the last two million years, surround intermontane lowland depressions of lower relief.

Many of the lowland basins are structurally-controlled, often formed between uplifted fault blocks. The Hawkswood Cut is constructed through one such low-relief depression, while the Ethelton Slip and the Mikonui earthflow both occur in foothill slopes flanking north eastern continuations of the Southern Alps.

Relief is extreme along the Southern Alps, and there are many peaks of 2,500-3,000m perpetually snow-covered. Mt. Cook, the highest peak in New Zealand, rises to 3,764m. Major drainage courses flow towards their respective coast lines off both flanks of the main divide. Within the study areas, the Hurunui, Waiau, and Conway Rivers transit the region in a south easterly direction and along often deeply incised antecedant drainage channels.

1.33 Climate

New Zealand has a maritime, temperate climate. Climate is influenced by location (the country lies between latitudes 34°S and 47°S), oceanic situation, and by the topography of its axial mountain system.

The southern Alps form a formidable barrier in the path of prevailing westerly winds which cross the southern Tasman Sea to reach New Zealand. Their effect is to produce contrasting climatic conditions west of the main divide, where precipitation as high as 5,000mm is experienced, from drier areas to the east.

The study areas, in general, usually experience warm summers with day temperatures often above 30°C, accompanied by dry, warm, often strong Foehn northwest winds. During winter, day temperatures are cool and winds from an easterly and southerly quarter prevail; frosts are common to low levels and most peaks over 2,000m are snow-covered. Rainfall over the year ranges between 700-2,000mm, and there is generally an increase in precipitation in the winter months (Fig. 3).

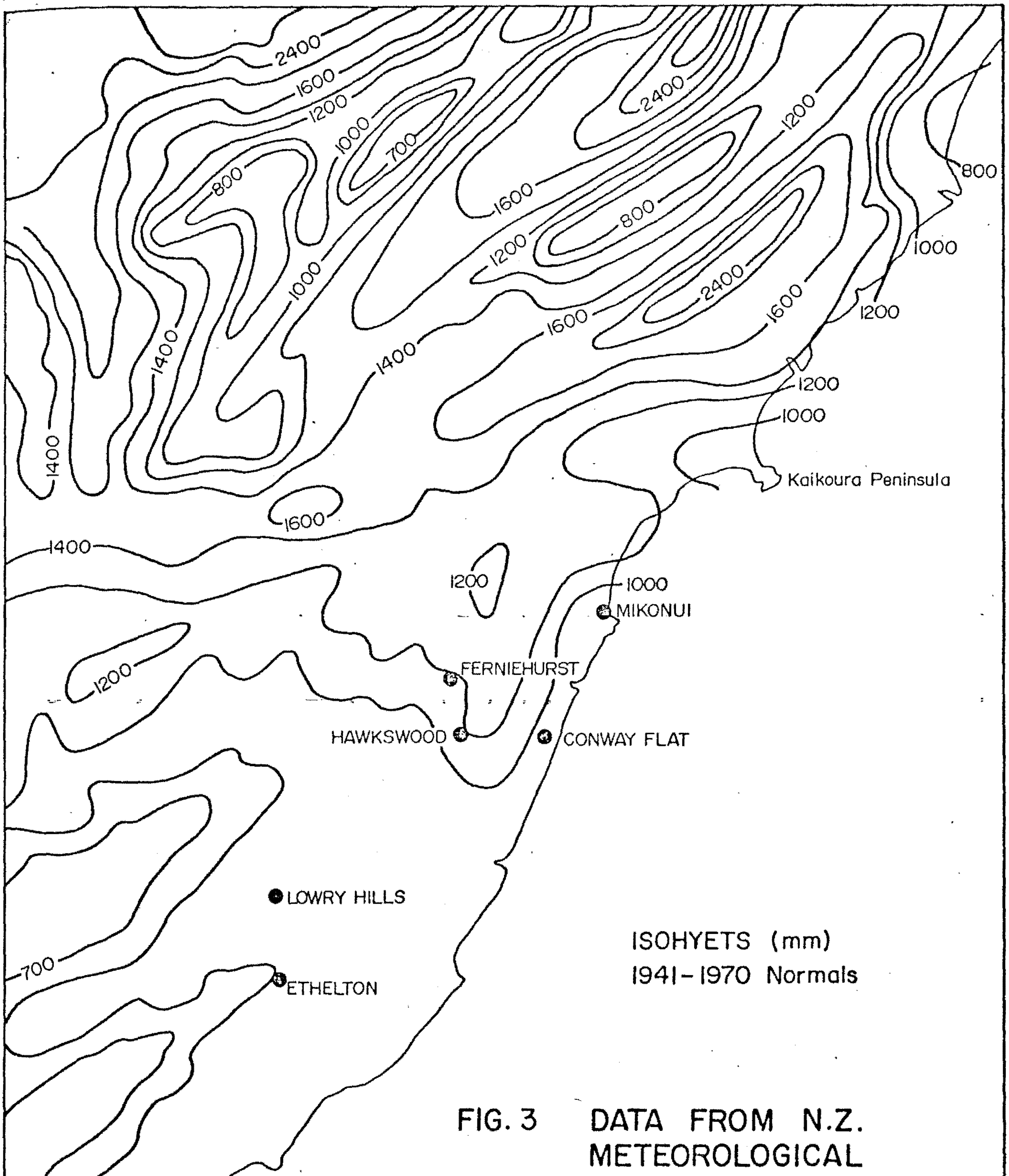


FIG. 3 DATA FROM N.Z. METEOROLOGICAL SERVICE

1.34 Vegetation

The native vegetative cover, mixed hardwood-podocarp-broadleaf forest, has been reduced to small isolated pockets throughout the areas. A modified flora, consisting of exotic grassland species (white clover-rye grass sward) intermingled with scrublands of gorse, bracken, manuka and broom, results from the introduction of European agricultural practices.

1.4 PREVIOUS WORK

In 1964 Gregg geologically mapped the study areas at a scale of 1:250,000 (Gregg, D. R., 1964, Sheet 18, Hurunui, 1st Edition). The Quaternary and late Tertiary geology has been detailed in a more recent D.S.I.R. mapping publication (Quaternary Geology - South Island, 1:1,000,000, 1st Edition, N.Z. Geological Survey Miscellaneous Series Map 6).

Warren and Speden (1977 in press) include the Mikonui earthflow and environs in an account of the Upper Cretaceous geology of the Haumuri Bluff district, southern Marlborough. Bell (1977) provides the first engineering geological account of the Mikonui earthflow.

Warren (in prep.) includes the Hawkswood Cut in a detailed study of the geology of the Leader Basin tectonic depression, North Canterbury.

An unpublished dissertation by Maxwell (1964) describes the stratigraphy and structure of the Kaiwara River district, the southeast boundary of which occurs two kilometres northwest of the Ethelton Slip. The two areas have many geological similarities.

Bell (in press) provides the only published account of the engineering and geological impact of Cyclone Alison on the coastal Kaikoura region.

Unpublished files prepared by the Public Works Department (now the Ministry of Works and Development) dating back to the beginning of this century, give details of the construction of the Main North Line. Unpublished Railways Department files furnished valuable accounts of the historical development of slope stability problems of the study areas, especially for the Ethelton Slip and Hawkswood Cut.

1.5 HISTORICAL

1.51 <u>Development of the Main North Line</u>

Construction of the Main North Line commenced from the southern (Addington) end of the line in 1872. Three years later development started from the northern terminal at Picton (Fig. 2).

Construction was undertaken by the Public Works Department as a staged development. By 1912 the southern section had reached Parnassus, while by 1915 progress in the north had reached as far south as Wharanui. These two sections were known at that time as the Parnassus and Picton Branch Lines, respectively. An inland North Canterbury line, known as the Waiau Branch, opened in 1882, met the Parnassus Branch at the Waipara Junction.

Further development did not proceed until the 1930's. The last section separating the Parnassus and Picton Branch Lines was completed in 1945. The railway has since been referred to as the South Island Main North Line.

The original 52 kilometre-long section of railway between Addington and Amberley (Fig. 2) was built to a 1,600mm (5'3") gauge and operated by the Provincial Government until November, 1876. Following that date the line came under the control of the then Public Works Department. In December, 1877 the gauge was reduced to the present 1,062mm (3'6").

The following table summarises the development of the Main North Line.

Addington-Parnassus: 1872-1912
(Ethelton Slip. Ethelton-Tormore section: 1906)
Picton-Wharanui: 1875-1915
Parnassus-Wharanui: 1939-1945
(Hawkswood Cut. Parnassus-Hundalee section: 1937;
Mikonui Earthflow. Hundalee-Kaikoura section: 1945).

1.52 History of Movement on the Ethelton Slip

During construction of the Ethelton-Tormore section of the Main North Line prior to 1906, it became apparent to the Public Works Department that the track would cross a landslide of considerable magnitude in a slope one kilometre east of the Ethelton Station. Plates 1-5 illustrate the nature of the slip in 1923.

Subsequent movements of the track and the slip at this section of the line have been recorded in unpublished Railways Department files dating back to 1907. The following are typical extracts from these records:

> "5.9.1907: Recent heavy rain has caused several heavy slips. At three places the slide is a continuous movement. Gang has been three weeks removing muck.

"12.8.1910: After remaining practically without movement for the last two or three years the slip at 23¾ miles (101km) is again showing signs of creeping towards the river. Movement started by heavy rains of last month, movement being about five inches in three weeks. About 4,000 cu. yds. should be removed from the slope to lighten the area. Work carried out.

"18.8.1910: The hill at 23.60m is still settling down and pushing the curve further out of line.

"26.7.1912: Hill and track creeping towards river faster than usual now that everything is so saturated with water. Track was pulled back 12 inches. Prior to this, track was pulled back nine inches in two weeks. Bad looking cracks in the face of the side cut above the railway. Track is curved at this place.

"21.8.1912: Track pulled back three feet ten inches and lifted one foot three inches. Total pull since middle of last month five feet seven inches.

"23.7.1916: Slip coming down over track. It consisted of a huge body of muck moving towards the line.

"29.9.1920: Since heavy rains started on 21.9.1920 the slip is on the move again. The track required the gang's attention every day in surfacing and aligning.

"14.8.1923: Movement noted over some 100 yards. Engineer attributes movement to the combined effects of the 25.12.1922 earthquake, in which innumerable cracks on the slip surface were opened up, and the heavy rains in May 1923.

"Sept.-Nov. 1923: A total of 335 feet of six inch earthenware pipes were layed in eight drives driven into the face beneath the track. The pipes were layed with a fall towards the river, and surrounded with loose rock.

"28.1.1924: A distance of 60 feet has been driven into this mass at river level, and no solid matter, except occasional boulders, can be found. The track is now as much as 12 feet from its original position.

"Sept. 1924: 18,700 cu. yds. removed from slip to lighten the area.

"10.11.1924: The slip is showing increased movement.

"26.1.1926: Fresh cracks of considerable magnitude were formed during the November, 1925 earthquake. There has been little rain since then to cause concern.

"12.6.1928: Slip appears to have moved two feet in recent weeks.

"Mid July, 1938: County road above track subsided three inches after heavy rain. A pronounced crack 50 feet long has developed in the centre of the road. Several large cracks have appeared on the face of the embankment, the toe of which is moving out towards the line.

"14.8.1945: Ethelton started moving again after heavy rain. County road above track has subsided six inches. Large portion of embankment has slipped into river 21 feet from track centre."

"1975: Following the passage of heavy rainfall over inland North Canterbury and coastal Kaikoura during Cyclone Alison in March, the slip appears to have started moving again following several years of inactivity. The track necessitated realignment once during the winter." (Railways observations.)

From these historical observations, the following conclusions concerning past movements on the Ethelton Slip have been drawn:

(a) The landslide is a natural phenomenon. Whether earth movements increased over the whole landslide generally following construction of the road and rail is not known. Localised activity in the lower (foot) region of the landslide adjacent to the road and rail in the years subsequent to 1906 is a result of the construction of the two routes.

(b) Movement was generally progressive, involving downward and outward movement of several large soil blocks, rather than the mass as a whole.

(c) The association of heavy rainfall with increased movement is well shown.

(d) Surface fracturing from slumping and earthquakes, enabling water infiltration into the slip to increase activity, has been a feature of the movement.

(e) Earthworks designed to lighten the slope have

Plate 1. "A is the rock face with small swampy lagoon at its foot (the rock face and swampy ground is still visible - writer). B is limit of cracks found in the Spring of 1923. C is the top of the main 1923 movement. C-D-E is the area of main movement in 1923", (1923 Railway's files).

PLATE 1

Plate 2. "F is channel cut out of face of slope and under ends of sleepers", (1923 Railway's files).

Plate 3. "C-D-E as before in Plate 1. G-E, face below track is slipping away into river. Track is 190 feet above bed of river. Flood level in May, 1923 was 30 feet above normal water level", (1923 Railway's files).

PLATE 2

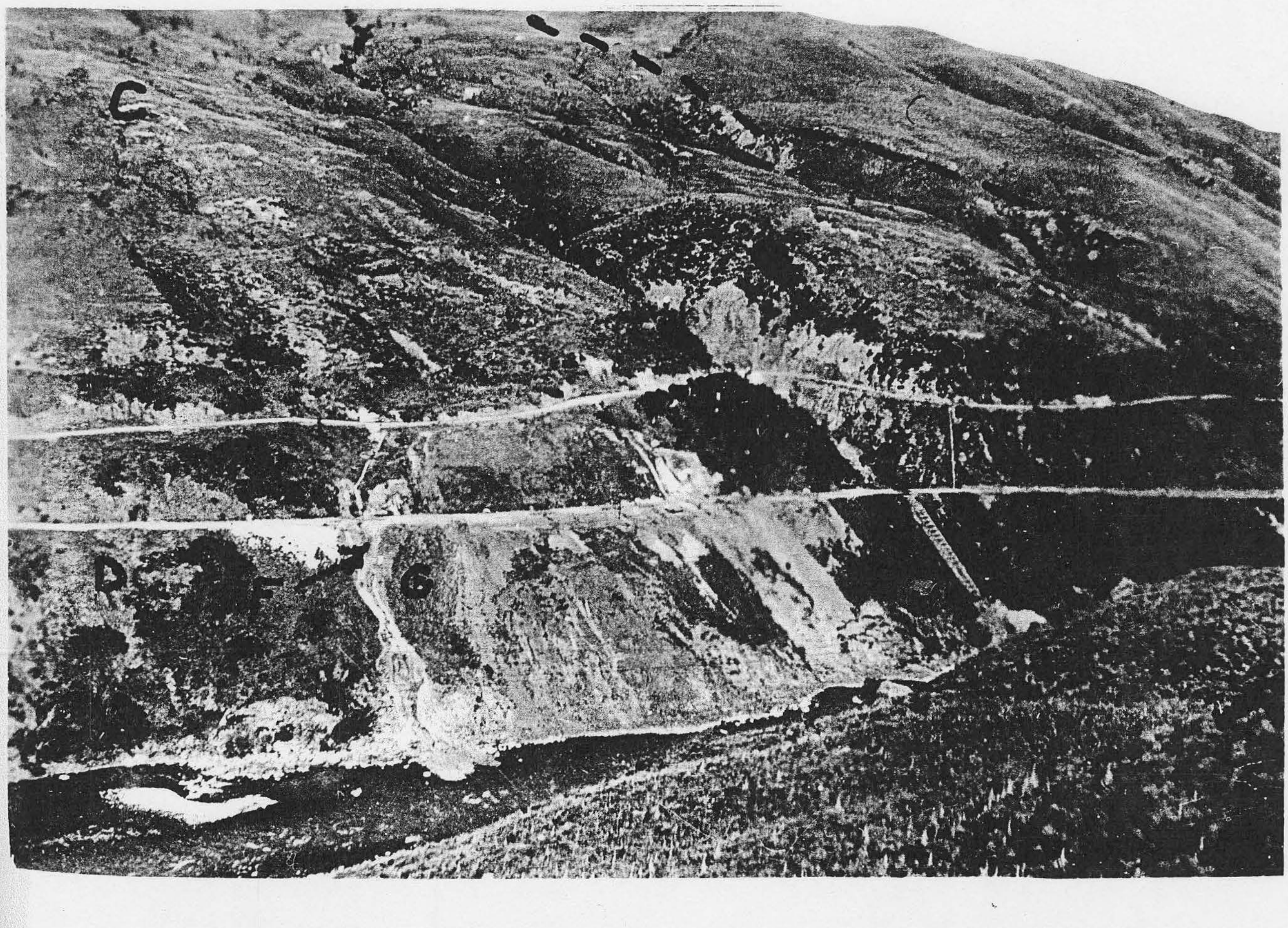


PLATE 3

Plate 4. "From C downwards may be seen the various principal slip faces. These are of varying ages but all comparatively recent", (1923 Railway's files).

Plate 5. Looking up the Hurunui River in 1923.

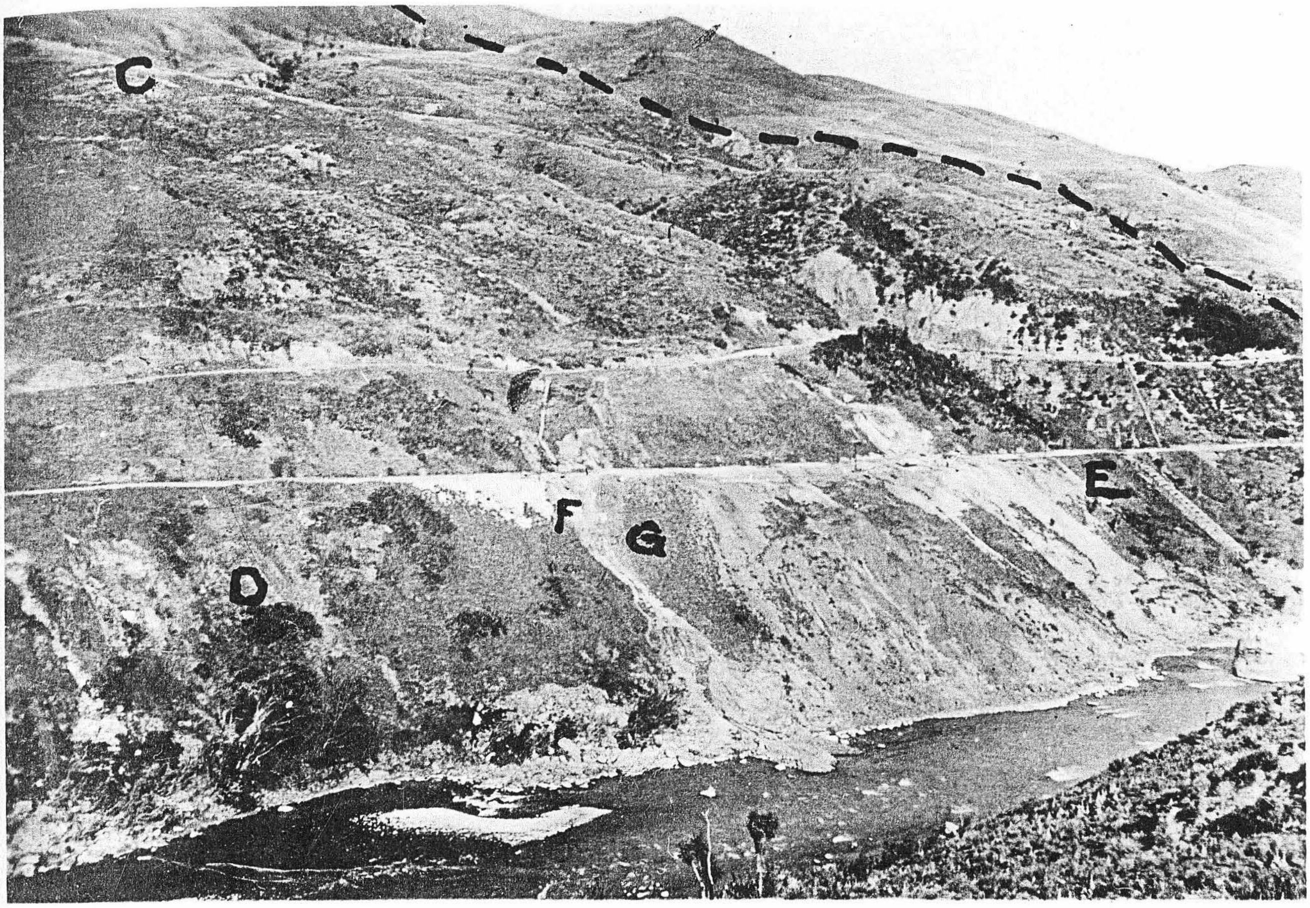


PLATE 4

PLATE 5

generally increased movement.

(f) Remedial drainage in the form of ceramic tile drains driven into the slope on a slightly inclined level were not successful. Dislocation of the drains by displacements within the slide material may account for this.

(g) Rates of movement during early parts of this century were often of the order of 0.5-1m per week.

(h) Both the Main North Line and the county road have been involved in the movement.

1.53 History of the Hawkswood Cut

The original cutting design, proposed prior to 1930, allowed for the excavation of two 520-metre approach cuts and a 520-metre tunnel. However, the original design was abandoned when test borings revealed that the excavation of a large-scale cut could be achieved faster and more economically. In 1930 a plan was initiated to excavate the Cut in three successive lifts working from both north and south approaches. By the time construction of the Main North Line ceased at the end of 1931, 40,500 cu. m had been removed from the southern approach.

After several years delay, authorization for the continuation of the Main North Line was given in 1936. By October of that year a 1.2 cubic metre Ruston steam drag-line commenced operation digging a gullett 10 metres deep and 20 metres wide along the length of the cutting. Two 0.6 cubic metre Diesel shovels commenced excavating on successive levels below the steam drag-line soon after.

By early 1937, the Ruston, along with a second steam drag-line, commenced widening the upper level, while a third 0.6 cubic metre Diesel lift-shovel commenced excavating at the intermediate level from the northern approach. Excavated spoil from the bottom lifts was hauled away by truck and train to form embankments, while material placed on top of the Cut was spread away from the final batter lines by a 6 and 9 cubic metre carry-all and an angle-dozer.

When the cutting was completed in October, 1937 a total of 289,031 cu. m of spoil had been removed, with a best total four-weekly output of 33,447 cu. m. During the latter part of the construction a total of 162 men worked three daily eight-hour shifts. Lighting during night shifts was supplied by a Diesel generating plant and carbide flares.

The problem at the Hawkswood, since its completion in 1937, has been the disruption to rail traffic from numerous earthfall and mudflow-type flandslides occurring along the length of the cutting. These slips, originating from the flanks of the batters, and generally small in nature, have occurred with such consistent frequency that corrective treatment has been under consideration for some 25 years.

Since the mid 1960's, disruption to rail traffic has been minimal, due mainly to improved machinery, more appropriate to deal with slips when they arise. Prior to the mid 1960's, blockages at this section of the line were considerable, and N.Z. Railway's files provide ample evidence of the exceptionally long delays experienced.

The earliest recorded slip occurred several weeks after completion of the excavation in 1937, delaying track laying for several days. During heavy rain in March and August, 1941 slips totalling 760 and 600 cu. m, respectively,

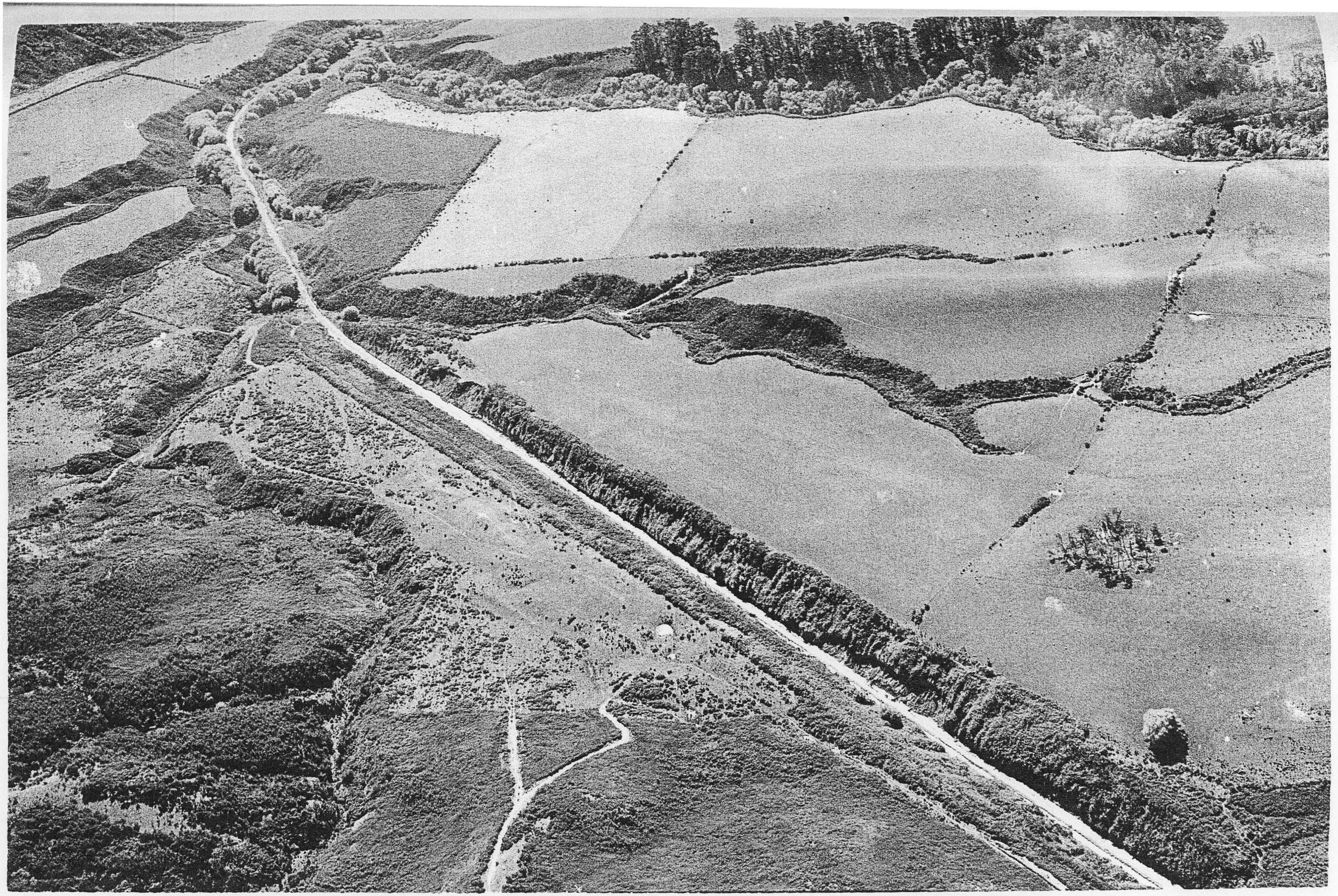


PLATE 6. The Hawkswood Cut.

caused delays to rail traffic, for eight days in the first instance, and covered the line to a depth of 1 metre in the second. Two slips in September and October, 1943 totalling 1100 cu. m caused a combined delay to traffic of four days. Heavy rain in February, 1945 caused several 300 cu. m slips of the order of 12m long by 2m deep, and 37m long by 0.75m deep, while in August of the same year the line was blocked by slips for three days. In August, 1961 a large slip of approximately 6000 cu. m filled the cutting after torrential rain. More recently 1100 cu. m of spoil were removed during 1975.

1.54 History of Movement on the Mikonui Earthflow

Unlike the Ethelton Slip and Hawkswood Cut, a record of significant movements has not been well documented. However, as with the landslide at Ethelton, historical observations indicate the Public Works Department were aware that the Main North Line would cross a large landslide between Claverly and Oaro, north of the No. 3 tunnel at Haumuri Bluff (Fig. 1). This is believed to be what is now known as the Mikonui earthflow.

Some of the more noteworthy extracts from Railway's files are recorded below:

> "7.8.1951: Slip moving inland from railway one mile, the width varies from one to eight chain. Original drains put in by Public Works Department appear to be still draining water off the top of the slip. Movement at railway eight inches per week.
>
> "1.8.1962: Considerable movement of recent weeks. Track was uplifted five inches and pulled back 14 inches which returns it to somewhere near its original alignment.
>
> "28.8.62: Track lifted four inches after having moved downhill five inches over a distance of 132 feet.

PLATE 7. The Mikonui earthflow. The landslide extends from the left upper centre to the lower right of photograph.

"24.7.63: A slip sliced from seaward side of track about one chain long and within seven feet of centre line. Track pulled back approximately 18 inches. 800 cu. yds. of fill placed as ballast."

Incidental to the subject of this thesis, but of considerable interest, are several areas in the vicinity of the earhflow in which some of New Zealand's early history is recorded. The remains of a recent Maori settlement is visible on the north western flank of the landslide, sited near the toe and overlooking the Mikonui Stream. There is also a Maori burial ground located between the railway and the south eastern boundary, 100m south of the toe. The old postal pony route (original state highway) traverses a ridge flanking the seaward boundary of the earthflow, approximately along the path of the track shown on the Site Plan (section 4). In addition, the grave of an early New Zealand whaler may be seen immediately below the seaward side of the railway several hundred metres south of the toe.

1.6 ACKNOWLEDGMENTS

Mr. J. K. Hill, Senior Lecturer in Engineering Geology, University of Canterbury, provided helpful and constructive supervision throughout all stages of the project, and critically reviewed the manuscript.

Grateful acknowledgment is made of the New Zealand Railways Department for their approval of the project, financial assistance with accommodation, and for transportation to the study areas.

Department of Scientific and Industrial Research, New Zealand Geological Survey, provided temporary employment over the 1975-76 summer field season, and made available equipment for monitoring of inclinometer access tubing installed at the Mikonui earthflow.

Central Laboratories, Wellington, and Christchurch Soils Testing Laboratory, Ministry of Works and Development, assisted with soils laboratory testing.

Investigations Section, M.W.D. (Christchurch), was responsible for all drilling investigations.

The assistance of the following people is also gratefully acknowledged:

D. H. Bell, Senior Lecturer in Engineering Geology, University of Canterbury;

I. R. Brown, Engineering Geologist, N.Z. Geological Survey;

T. C. Clisby, Assistant Engineer, N.Z. Railways Department;

T. Dodd, Senior Lecturer, School of Engineering, University of Canterbury:

A. Downing, Technician, Department of Geology, University of Canterbury;

D. D. Gordon, Senior Lecturer, School of Engineering, University of Canterbury;

R. Hall, Technical Officer, School of Engineering, University of Canterbury;

G. Hunter, Assistant Engineer, N.Z. Railways Department;

L. Leonard, Draughtswoman, Department of Geology, University of Canterbury;

D. McConchie, Student, Department of Geology, University of Canterbury;

G. Mansergh, Engineering Geologist, N.Z. Geological Survey;

A. Nicholas, Technician, Department of Geology, University of Canterbury;

N. D. Perrin, Technician, N.Z. Geological Survey;

J. Rutledge, Special Projects Engineer, M.W.D.;

G. Salt, Engineer, M.W.D.;

G. Warren, Geologist, N.Z. Geological Survey;

L. Wilson, Engineer, N.Z. Electricity Department.

Mr. G. K. Grocott is thanked for critically reviewing the manuscript, and appreciation is expressed to my wife, Janey, and Mrs. Eileen Paterson, for typing the first draft.

The writer was a recipient of a University Grants Committee, McKee Trust Postgraduate Scholarship during 1976, awarded annually to a New Zealand MSc geology student.

SECTION 2: THE ETHELTON SLIP - A LANDSLIDE IN COLLUVIUM

2.1 INTRODUCTION

A large proportion of the mountainous, alpine areas of the South Island's Southern Alps are underlain by hard, indurated greywacke-type sandstones and mudstones (argillites) and their metamorphic (schist) equivalents. The sandstones and argillites, and related volcanic intrusives, are known collectively as Torlesse Supergroup sediments.

In many districts Torlesse rocks extend east and westwards from the main divide towards the New Zealand coastline, appearing as inliers in the form of small mountain ranges between relatively low relief landforms of soft-rock, consolidated Tertiary marine sediments and younger alluvial outwash surfaces.

Slopes formed in Torlesse Supergroup sediments are typically steep, commonly ranging 25^{o} to the near-vertical. Sediments exhibit extremes of mechanical and chemical weathering, from rocks in which slight discolouration and loss of strength occur, to residual soils. In many areas of extreme relief, peaks of hard, fresh rock, stripped bare of soil and vegetation, form above large scree and talus deposits.

Mass wasting of slopes formed in areas underlain by greywacke sandstones and argillites is principally one of two types:

(a) Failure of completely weathered or residual soil where stability is controlled by the shear strength of the material. A conventional soil mechanics approach is usually adopted for analysis.

Failure in weathered rock slopes is usually governed by the development of at least several metres of soil mantle, generally on slopes less than 35^{o}.

A subdivision of failures in completely weathered and residual greywacke soils is commonly made:

(i) Rotational or circular arc failures usually associated with cuts excavated for engineering structures, where slow continuous or intermittent movement is involved.

(ii) Rapid debris slides and avalanches fail along shear surfaces within the soil mantle, or at the colluvium-bedrock interface. A reduction in the effective normal stress in the slope material above a bedrock hydraulic discontinuity during high intensity or prolonged rainfall is the usual triggering mechanism. The ambient conditions leading to failure may be a combination of vegetational changes, the location of roads and railways, or oversteepened slopes.

(b) Failure of slightly weathered or fresh rock slopes where stability is controlled by the orientation and strength of discontinuities. Failure of the rock mass occurs by sliding along a single facture (plane failure), or, more commonly, along two or more intersecting discontinuities (wedge failure). In steep alpine areas lacking vegetation, frost and ice wedging along closely spaced fractures may be the cause of large scree and talus deposits.

In natural slopes, structure-controlled failures occur principally in areas of high relief or on slopes over 35° where shallow regolith depths are common.

For moderately or highly weathered rock slopes, intermediate between fresh rock and true residual soil, stability is controlled by both material strength and geometry and strength of discontinuities. Rotational slips and structure-controlled slides may both occur.

This section describes the Ethelton Slip, a landslide in a natural slope formed in weathered Torlesse Supergroup rocks. The landslide involves a laterally extensive landmass in a slow, intermittent, downhill movement in which deep weathering of a basic volcanic bedrock instrusive into Torlesse sandstones and argillites appears to have played a fundamental role in the failure.

The Ethelton Slip lies 33 kilometres north, and 18 kilometres south, of the North Canterbury townships of Cheviot and Waipara (Fig. 4). The townships function as service centres to the surrounding farming districts. One kilometre to the northwest is located the slip's namesake, Ethelton, an infrequently used railway station and almost abandoned settlement with a current population of one.

The toe of the landslide borders the true left bank of the Hurunui River, midway between the river's confluences with the Kaiwara and Greta Rivers. This section of the Hurunui has typically steep-sided slopes above the river, and is known locally as the Ethelton Gorge.

The Hurunui River, one of several west-to-east flowing North Canterbury rivers draining the eastern flanks of the South Island's Southern Alps, continues a further 16 kilo-

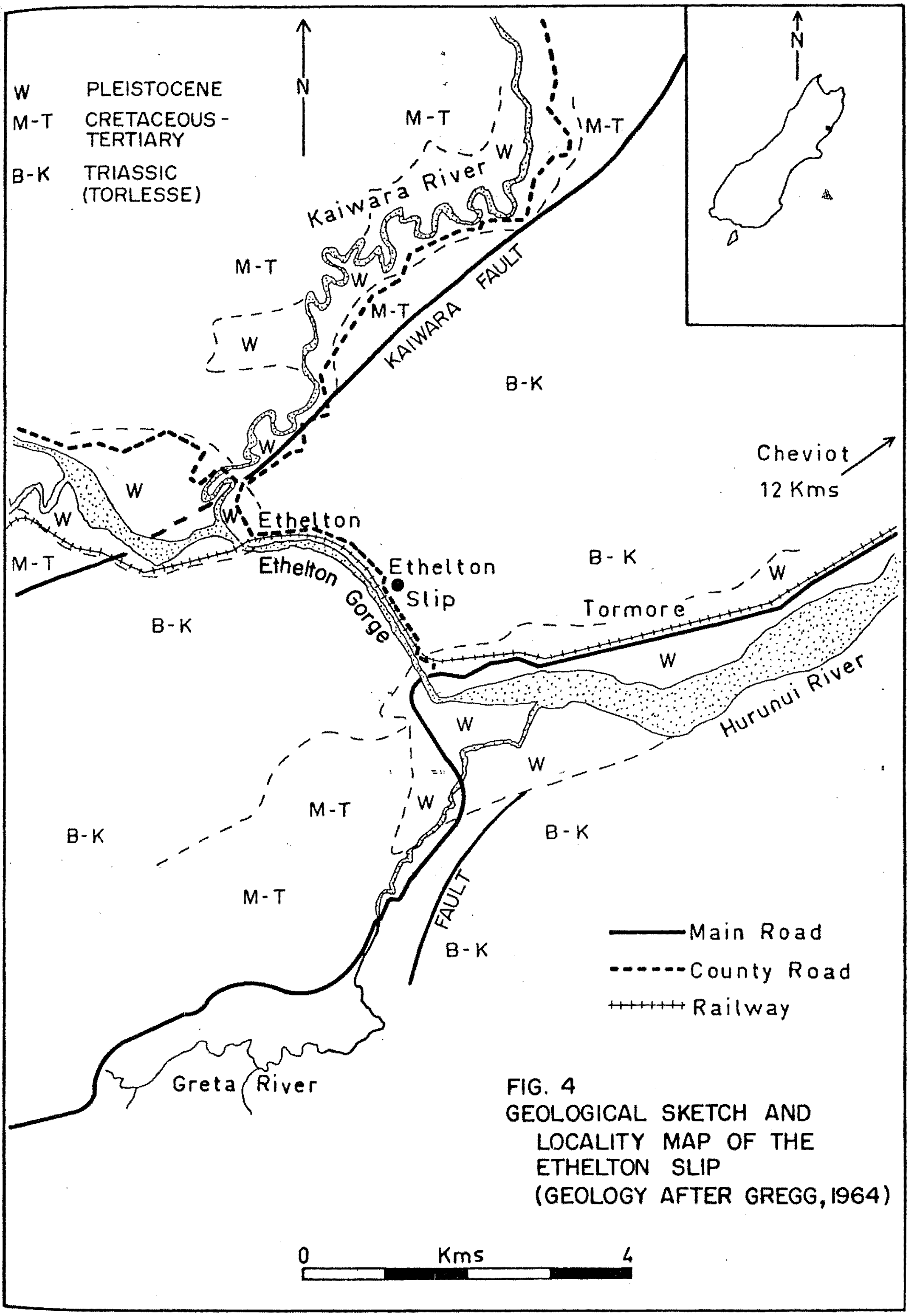


FIG. 4
GEOLOGICAL SKETCH AND LOCALITY MAP OF THE ETHELTON SLIP (GEOLOGY AFTER GREGG, 1964)

metres from the site before reaching the South Island's east coast.

Running parallel with, but higher and back from the Hurunui, the Main North Line and a Cheviot County Council road both cross the toe of the landslide. One kilometre southeast of the site, the Main North Line leaves its companion Christchurch-Picton State Highway No. 1 to travel inland across the toe of the slip before rejoining the highway some thirteen kilometres further south.

Land use at the site, other than that taken up by the location of the two transportation routes, is restricted to livestock grazing.

The limited time available for this thesis precluded detailed site investigations being undertaken at all three of the study areas. As a result, the Ethelton Slip has received less attention than would normally be required for a landslide site investigation such that stability correction measures could be confidently undertaken. Therefore, section 2 only examines the postulated failure mechanisms, summarises field investigations so far undertaken at the site, and recommends further studies necessary to permit a slope stability analysis to be computed. Section 5.31 proposes remedial measures based on the investigation programme so far carried out.

In conjunction with this section, the following plan should also be examined:

"Site Investigation Plan of the Ethelton Slip".

2.2 METHODS

A preliminary review of the documentation was under-

PLATE 8. The Ethelton Slip. The landslide occupies the basin-like feature in the centre of photograph. The Main North Line and the Cheviot County Council road are clearly seen at the foot of the landslide.

taken. This consisted of a study of Railways Department files dating back to 1907. Several 1923 photographs of the landslide were also located (Plates 1-5). Results of this documentation review show the slide has a history of continuing downward and outward, intermittent movement. Displacement of the railway towards the Hurunui River accompanied this movement.

The initial studies also included a reconnaisance of the site using sterographic projection of vertical air photographs. One series of photographs was viewed:

run numbers	date
1814/44,45,46	12.8.1950

In addition, a series of oblique aerial photographs was taken by the writer during September, 1976 (Plates 8-9).

A base map of the site was executed at a scale of 1:1000 using a Wild T16 theodolite and staff. Mr. Mervin Harvey, assistant engineer, N.Z. Railways, assisted in the field.

Topographical spot heights obtained from the theodolite survey were reduced by means of a FOCAL (formula calculator) computer programme written for use on the School of Engineering, University of Canterbury's PDP-8 computer. The programme allowed all spot heights to be assigned north and east coordinates and reduced levels. The method enabled simple plotting of heights on to a grid-coordinate base plan.

Topographical contours at 5m intervals were traced on the base map. All relevant geomorphic data, including slope angles, road, rail, streams, ponding and seepage, were noted during the initial survey or added subsequent to completion of the base map.

The site was geologically mapped at the same scale as the base map. Exposures were generally poor, restricted to a single stream channel, the county road, railway, and a few outcrops at the head, toe and margins of the slide. Geological information collected included noting bedrock and soil types, a study of weathering sequences, delineating zones of active slope movements, and observing the progressive development of tensional cracks in active slope zones.

Eight surface marker stations were installed, five inside and three outside the potential area of movement, and precise surveying of markers carried out to determine rates of movement. A rainfall gauge was monitored to correlate precipitation with movement.

Two seismic refraction traverses using a dual-channel seismograph were undertaken to calculate depths of colluvium. An incomplete laboratory testing programme on disturbed soil samples obtained from exposures completed the field investigation.

2.3 SITE DESCRIPTION

The site is located above that portion of the Hurunui River which is referred to locally as the Ethelton Gorge. The Hurunui River, within the Ethelton Gorge, extends from the river's confluences with the Kaiwara and Greta Rivers (Fig. 4). Throughout the gorge, the Hurunui River is narrow (50-70m wide), and swift flowing. Above the gorge, slopes rise steeply (up to 45°), with total relief between valley floor and slope crest of some 350m (Plates 8 & 9). At the site, the valley floor is located at an elevation of approximately 60m.

The toe of the Ethelton Slip forms a 450m long portion of the true left bank of the Hurunui River, approximately midway through the Ethelton Gorge. The site has an aspect towards the southwest, covering an area of 30 hectares. Relief between valley floor and the upper boundary of the site is 200m, though the crown*, immediately above the head of the landslide, lies some 150m below the slope crest. The horizontal distance between the valley floor and the upper boundary of the site approaches 650m, while that between the two lateral boundaries approaches 450m. The width to length ratio is therefore 0.69.

Oblique aerial photographs (Plates 8 & 9) illustrate the semi-circular, basin-like physiography of the landslide; prominent ridges define both the northwest and southeast boundaries of the site. The physiography of the site is such that the slopes surrounding the upper and lateral boundaries of the landslide naturally facilitate surface water runoff towards the centre of the site. At the same slope location, relief of 40m exists between the centre and margins of the landslide.

Slopes forming the margins of the landslide, and above the upper and lateral boundaries, are steep (ranging to 45^{o}); similar slopes prevail at the extreme head and toe of the site. In the central regions of the site slopes are gentle (typically $10-20^{o}$), and in one area are marginally reversed, causing ponding during the wet.

* Crown refers to insitu material, immediately above the head of a landslide.

PLATE 9. The Ethelton Slip.

PLATE 10. Tension cracks. Scale is 19cm high.

An unnamed stream bisects the site, incised up to 10m at a point above where the stream passes beneath the county road. The stream originates at the crest of the slope 150m above the upper boundary of the site.

A large knoll, or topographic high, above the county road, is prominent in oblique aerial photographs (Plates 8 & 9) southeast (on the true left side) of the stream bisecting the site. Within the stream, indurated volcanic bedrock is seen underlying the knoll. For a distance of 200-300m southeast of the unnamed stream, instability within the slopes forming the knoll have caused considerable road maintenance problems in recent winters. Above the road, surficial debris slides and flows terminate on the road during heavy rainfall. Below the county road, undermining and slumping have brought the actual road foundations into jeopardy on occasions. Attempts by the Cheviot County Council to prevent undermining and slumping of the road margins by the installation of crude tied-back retaining walls have been largely unsuccessful. Instability within the slopes forming the knoll have not affected the railway in recent years.

A second area of slope instability, believed to be of a deeper seated nature, has been mapped in a region above and below the county road, 100-250m northwest of the unnamed stream. During the period of this study, the first evidence of slope instability at this locality was observed over the 1975-76 summer, when a series of dry, open tensional cracks were mapped in the region of the No. 7 survey station. Subsequently, the progressive development of several large surface fractures and cracks took place over the winter of 1976 (Plate 10).

In the field, these cracks are commonly continuous over many metres, and up to 0.3-0.4m deep. Throughout the 1976 winter the cracks were water-filled. At one location, downhill movement of a small soil block produced a semi-circular inclined scarp at its head, up to 0.5m wide as the crack progressively opened. Further downhill the scarp tightens and grades into a zone of shear. Slickensides in both the scarp and the shear zone indicate movement.

A visit to the site during August, 1976 resulted in the discovery of two tensional features in the metalled surface of the county road. The cracks were tight but well defined, separated by some 30m, and were curved in form. It is inferred the cracks are a continuation of each other, meeting at a point higher in the slope. The up and downslope continuation of the features could not be traced. A small, though noticeable, depression of approximately 2-3cm could be observed in the road separating the two. Subsequent visits to the site revealed no further development of the features.

The No. 4 survey station, which is located 1m above and back from the railway, occurs immediately below that section of the county road in which the tensional cracks aforementioned occur. The railway in the vicinity of the No. 4 station, involving about 100m of line, necessitated realignment once during the 1976 winter. No other section of track at the site required this treatment during 1976.

The distribution of tensional cracks and shear zones mapped at the site indicates a zone of deeper seated slope movements extending from the railway, at the No. 4 survey station, upslope to a point slightly higher than the No. 7 marker. The active zone extends both left and right of the line between the two stations for several tens of metres (see Site Plan). Both road and rail pass through the active zone.

With the exceptions of the two relatively small areas of slope instability previously described (that is, the zone of slope activity in which shear and tension cracks were recorded, and the area of surficial slumping and sliding associated with the county road), no other evidence of land-sliding, either of a shallow or deeper seated nature, were recorded during the time of this study. The results of engineering geological mapping to date, therefore indicate that movements at the site are localised, and are not occurring generally over the landslide as a whole.

Observations on the unfavourable foundations upon which the railway is constructed cannot be ommitted. Under-lying the ballast over a considerable distance is a layer of slag, the products of boiler waste from now-disused steam locomotives. This material does not consolidate and is noted for its loose packing. An attempt to provide suit-able track foundations should be made.

At the commencement of this study, unsatisfactory railway foundations were initially considered by some Railway personnel as a possible cause of recent track move-ments in the vicinity of the No. 4 survey station. However, in light of the fact this section of line occurs within the zone of active slope movements described previously, founda-tion settlements are not believed to be a reason for continuing track displacements at this location.

2.4 GEOLOGICAL SETTING

2.41 <u>Regional Geology of the Site</u>

2.41.1 Pre Cretaceous Rocks. The oldest rocks exposed in the Ethelton Gorge and Kaiwara River districts are geosynclinally-derived marine and non-marine sediments, principally undifferentiated greywacke sandstones and argillites. Minor volcanics, and their associated cherts and jaspillites, and conglomerates and limestones also crop out. The name Torlesse Supergroup is applied generally to these rocks (Fig. 4). On the basis of sparse fossil evidence, Torlesse rocks in the Ethelton Gorge district have been assigned a Triassic age (Balfour to Kawhia Series). Immediately to the west of the Kaiwara River, the Torlesse assemblage have a Jurassic age (Herangi to Oteke Series), (Gregg, 1964.)

Greywackes and argillites are typically well bedded in the Ethelton Gorge, Kaiwara River districts, both graded and alternating discreet bedding recognised. Sandstone beds are hard, indurated, generally massive, and more resistant to weathering than argillites; these properties produce a crenellated profile between well bedded, alternating sequences. Argillites are only partially indurated, commonly to the extent of other younger New Zealand marine mudstones of Teriary age.

Volcanic rocks in contemporaneous association with greywackes and argillites are common throughout the district. These rocks occur as lavas, resulting from extrusion onto the sea floor, and as dikes injected into unconsolidated sediments. Included in the volcanic assemblage are basaltic lavas, spillites, tuffs, jaspillites and cherts. Red, occasionally green, haematite-rich spillites are distinguishable from dark, greenish-black basaltic lavas.

Minor bioclastic limestones, breccias and conglomerates, complete the Torlesse assemblage. A prominent conglomerate, the Ethelton Conglomerate, is exposed in outcrops in the Hurunui River at the site of the old suspension bridge several kilometres upstream from the site. The conglomerate consists of poorly sorted to well sorted pebbles of granite, rhyolite, greywacke and argillite in a sand matrix (Maxwell, 1964). The deposit, bedded in places, is lensoid in shape and continuous over several hundred metres only.

The abundance of calcereous cannon-ball concretions within Torlesse sedimentary sequences is a feature of the Kaiwara district. These may reach 1m in diameter, though 0.2-0.3m is more common.

Greywacke sandstones and argillites, and their related extruded and intrusive volcanics, form the rocks at the site of the Ethelton Slip.

2.41.2 Late Cretaceous to Tertiary Rocks

Where rocks of the Torlesse assemblage are overlain by younger sediments, a marked angular unconformity defines the contact. This hiatus is recognised generally throughout New Zealand as a period of erosion and peneplanation of the Torlesse surface prior to a number of marine transgressions and deposition through Late Cretaceous and Tertiary times.

Late Cretaceous and Tertiary rocks are not preserved at the site. These sediments occupy fault-angle depressions and are exposed one to two kilometres immediately to the east and west. A vast range of lithologies, including basal carbonaceous sands, glaucontic sandstones, calcereous siltstones with concretionary bands and conglomerates, reflect changing depositional environments from fresh water, to lagoonal, through to deep marine.

Generally the rocks are consolidated, though not to the extent of being indurated.

Dune sands of Hawera age (Pleistocene) are exposed above both banks of the Hurunui River, one kilometre south-east of the site.

2.41.3 Regional Structure. The major structural feature of the district, the Kaiwara Fault, is northeast-trending, running parallel with but slightly to the east of the Kaiwara River. Traceable for many kilometres to both the northeast and southwest, the fault is one of the principal tectonic features of North Canterbury.

The Kaiwara Fault is predominantly reverse, with the south easterly dipping hanging wall vertically displaced some 500m since the Late Cretaceous (Maxwell, 1964).

Uplift on the Kaiwara Fault has caused the formation of two tilted fault blocks: to the west of the fault, the Lawry Peaks block; to the east, the Greta Peaks block. These fault blocks consist chiefly of sediments of the Torlesse Supergroup. Late Cretaceous and Tertiary rocks are preserved in the fault-angle depression separating the two.

The Kaiwara Fault is still regarded as active. An earthquake on 25 December, 1922 is thought to have originated at depth on the fault. As a result of the earthquake a large landslide blocked the Waikari River, a tributary of the Hurunui River, upstream from their confluence. A large body of loose debris, 30m wide by 10m high, dammed the river, bringing the safety of the railway in the Ethelton Gorge and the Hurunui railway bridge into jeopardy. A drain cut in the right bank of the dam allowed the gradual erosion of material without catastrophic failure (N.Z. Railways Department files).

Maxwell (1964) reports the occurrence of a surface fracture following the Cheviot earthquake of 11 January, 1951.

Several smaller branch and splinter faults cross the district. These strike northeast, consistent with the regional strike, and both normal and reverse faults occur.

Structure of Torlesse rocks in the Ethelton Gorge, Kaiwara River districts appears simple, involving near-vertical tilting along a consistent northeast-trending strike, and is certainly less complex than the highly folded and faulted Torlesse structure usually encountered elsewhere in New Zealand. As a structural comparison, Petrie (1974) describes an area of chaotic melange in Torlesse rocks in the Studleigh Range, 80 kilometres west of the site. Three structural zones, bedded rocks, disrupted strata, and chaotic breccia, are recognised.

Torlesse Supergroup sediments are inferred to reach a thickness of at least 10-12,000m in the North Canterbury region generally.

Folding, possibly related to Tectonic drag effects, is common in late Cretaceous and Tertiary beds adjacent to the main faults.

2.42 Geology of the Site and Environs

2.42.1 Stratigraphy. Torlesse Supergroup sediments of Triassic age form the rocks underlying the site. Of the various Torlesse lithologies only undifferentiated greywackes and argillites and their related volcanics are exposed.

Poor exposures limit descriptions of sandstones and argillites to two areas around the margins of the landslide, an outcrop in the Hurunui River 700m downstream, and in the slope adjacent to the site on the opposite side of the river.

At the toe of the slip, massive, insitu sandstone occurs as water-smoothed knolls. These rocks can be traced at river level for 200m. Similar massive sandstone is located halfway up the slide's south eastern margin. Here the rock has a slightly weathered appearance.

The most detailed exposure in the vicinity of the site is located 700m downstream on the left bank of the Hurunui. A water-level-measuring station is founded on these rocks. Well bedded, light gray sandstones are seen alternating with dark gray to black argillites. Sandstones are typically hard and massive. Bedding thicknesses range from 0.3-1.0m. A crenellated profile due to differential weathering of the two lithologies is also weakly developed.

A farm track adjacent to the site on the opposite side of the Hurunui River has revealed sections several hundred metres long comprising well bedded, alternating greywackes and argillites. At this locality, beds are typically 0.1-0.3m thick. Both lithologies are highly fractured.

Sedimentary features within sandstones and argillites such as flute and load casts and cross bedding were not studied, as these have no engineering significance. Calcereous cannon-ball concretions were not observed.

Outcrops of basic volcanic bedrock at the site occur more frequently than those comprising sandstones and argillites. These rocks are spillites, red in colour, though occasionally green. At two locations at the site, in the unnamed stream bisecting the landslide (at the level of the county road), and at the head of the landslide, spillites extend over some 30-40m. At the former locality, unweathered volcanic bedrock appears hard and non-vesicular, though the rock mass is fractured; at the latter, the rocks are veined, moderately weathered, and appear brecciated.

A third exposure of volcanic bedrock occurs on the opposite bank of the Hurunui River, adjacent to the south eastern margin of the landslide toe. All three exposures of spillitic bedrock aforementioned appear to lie on a straight line; whether the sediments form a continuous body of volcanic bedrock at depth beneath the soil cover of the landslide is not known. The three localities possibly represent areas of lava extrusion onto the sea floor.

Dikes of spillitic lava, intrusive into sandstones and argillites, occur frequently within exposures adjacent to the site on the opposite side of the Hurunui River. Injection of dikes took place contemporaneously with sediment deposition, producing long lense-like bodies, parallel to beds, up to 0.3m thick. The contact between intrusion and sediment is sharp; invaded beds commonly have a baked appearance.

In handspecimen, spillites have a typical dark reddish brown, iron-stained appearance.

Recent alluvium forms 7-8m high banks along the unnamed stream bisecting the landslide in the mid to upper regions of the site. Considerable proportions of slope wash debris are mixed with the alluvium.

A mantle of weathered, redeposited soil and rock particles overlies those portions of the site not exposed as bedrock or alluvium. A slope mantle therefore covers the largest portion of the landslide surface. This redeposited mantle has been derived from weathered Torlesse Supergroup bedrock assemblages (both sedimentary and volcanic) underlying and surrounding the landslide. The regolith has been transported to the site from its place of origin by gravity; colluvium would therefore be the correct terminology for the slope mantle covering the site.

Colluvium comprises both undifferentiated sandstone and argillite and volcanic detritus, covering all grain size grades from large boulders to fine silts and clays. Generally the colluvium is non plastic.

As with many other slopes underlain by a true colluvium, the soil mantle covering the site is a result of the weathering of the underlying, oversteepened bedrock. Downcutting and undercutting of the bedrock by river action caused the slope to become oversteepened, while downslope mass movements of the products of weathering formed the colluvium. Colluvium therefore comprises the material involved in the movement at the site; a more detailed account of the weathering and landsliding processes is given in sections 2.43 and 2.5.

The age of the colluvium at the site is uncertain.

Much of the regolith is thought to have formed within the last 10-14,000 years, during which the Hurunui River (along with many other New Zealand rivers) underwent rapid downcutting.

2.42.2 Geological Structure. Bedding in Torlesse sandstones and argillites is preserved only in areas beyond the landslide. Downstream of the site, beds are typically near-vertical to slightly overturned, striking northeast-southwest. The strike is consistent with the regional trend in the Ethelton Gorge, Kaiwara River districts.

However, the attitude of bedding is unimportant in relation to the stability of bedrock underlying the site. The presence of discontinuities in general (fractures, joints, bedding) throughout the rock mass is regarded as more important, allowing more of the rock surface to become exposed to the weathering and slope forming processes. The extent of fracturing of both Torless lithologies (sedimentary and volcanic) beneath the site is unknown, though the Torlesse assemblage adjacent to the site on the opposite side of the Hurunui River are moderately to highly fractured.

No faults were mapped in the environs of the site. However, seismic activity (for example in 1922) produced ground fracturing on the landslide surface. This was due to soil shaking rather than to fault displacements in the vicinity of the site. The result of surface cracking, in the years following 1922, was to allow infiltration of water to penetrate the slide mass causing rates of movement to increase.

2.43 Weathering

Soils formed as a result of weathering of a natural bedrock slope are classed as either residual or transported. Where the bedrock slope is gentle (typically less than 20^{o}), chemical and mechanical weathering of the bedrock surface may proceed insitu such that a gradational sequence of layers with differing physical and chemical properties result. The weathering profile will be bounded by residual soil at the surface and by unweathered or fresh bedrock at depth. A gradual decrease in soil shear strength with increasing degree of weathering, may be the most significant physical propertyto accompany insitu weathering of bedrock; as a result, an almost imperceptible downslope creep of the residual soil at the surface is the usual mechanism of mass waste removal of insitu weathered bedrock.

Where the bedrock slope is oversteepened, either through downcutting or undercutting of the slope by river or wave action, instability of the residual soils result. The decrease of the slope angle to an equilibrium value takes place by intermittent landsliding. As a result, the products of weathering become involved in gravitational mass movements. Unlike slopes formed through insitu weathering, the interface between weathered, transported soils, and the unweathered bedrock, is sharp. Furthermore, the properties of weathered, transported soils will change considerably over short distances due to erratic variations in the degree of weathering (Terzaghi and Peck, 1967).

Weathering of oversteepened, natural bedrock slopes results from a number of different, though interacting, processes. Initially, the bedrock may become oversteepened due to the downcutting or undercutting action of rivers or waves at the toe of the slope. As a result of a rock stress-unloading process accompanying the downcutting, sets of stress relief fractures and joints develop in the near-surface bedrock profile. With increasing depth below the surface, these stress-relief-induced discontinuities become tighter and less persistent.

Weathering of an oversteepened bedrock surface will proceed by mass movements within the incompetent near-surface fractured rock. The rate of disintegration will depend on the degree of fracturing of the rock mass. However, the rate of weathering will be aided by pore pressure and temperature variation within the fractures, as well as by a general loss of rock shear strength as a result of chemical alteration and mineral replacement of the near-surface rock. Chemical alteration may be an important addition to the general weathering process if the rock mass contains soluble constituents (for example, calcerous sediments), or if the mineralogy of the rock mass is susceptible to replacement (for example, iron or magnesium-rich sediments).

The Ethelton Gorge comprises that section of the Hurunui River between the river's confluences with the Kaiwara and Greta Rivers (Fig. 4). The Ethelton Slip is sited approximately midway through the gorge. Outside of the study area, the slopes rising steeply above both sides of the Ethelton Gorge exhibit longitudinal convexo-concave slope profiles. Such profiles are characteristic of areas comprising sandstones and argillites of the Torlesse

Supergroup, in which landsliding has resulted in a relatively thin (2-3m thick) cover of weathered, transported slope mantle. These slopes have now attained equilibrium, and are believed to have formed from numerous surficial debris slides and flows.

However, at the site of the Ethelton Slip, the slope profile is quite out of character with those profiles typical of slopes above the rest of the gorge; in fact, the slope profile at the site (the head of the landslide has a sunken appearance while the toe of the slope is heaved) reflects a slope in which deeper seated, possibly rotational movements are occurring. As well, the basin-like physiography (Plates 8 & 9) of the site (producing relief up to 40m between the centre and margins of the landslide), indicates that the depth of bedrock removed through weathering is considerably greater than in other areas of the Ethelton Gorge outside of the study area. Furthermore, recent surface fracturing, and monitoring of movement on surface markers (section 2.7), indicate the landslide has not yet attained equilibrium.

To account for these differences of slope profile and depth of weathering (that is, between the Ethelton Slip and those slopes outside the study area above the rest of the gorge), it is the opinion of the writer that a process of differential weathering has been, and still is, in operation within bedrock slopes above the Ethelton Gorge.

Slopes outside the study area are underlain principally by undifferentiated sedimentary lithologies (sandstone and argillite). As a result, the weathering processes forming the relatively thin veneer of transported slope mantle are principally due to the mechanical disintegration of the underlying fractured bedrock surface.

At the study area, the material underlying the site is believed to comprise a spillitic bedrock, the rock being one of several volcanic associations of the Torlesse assemblage (section 2.42.1). Although no mineralogical analyses were undertaken by the writer, spillites are known from other Canterbury localities to contain ilmenite, pyroxene (commonly titaniferous augite), sodic feldspar and calcite. Through chemical weathering, pyroxenes may be replaced by epidote, chlorite or actinolite, and sodic feldspars may weather to sericite or be replaced by chlorite. The weathering products, haematite or goethite, occur as an opaque mineral throughout the rock. Spillites are therefore particularly susceptible to chemical weathering and alteration; a general loss of rock shear strength accompanies chemical alteration of the rock mass.

X-ray diffraction analyses performed by the writer on two weathered spillitic soil samples revealed the presence of a swelling chlorite, epidote, calcite and feldspar, possibly albite (section 2.8).

Weathering of the volcanic bedrock surface underlying the landslide, to a depth considerably exceeding bedrock weathering outside of the site, therefore resulted from a combination of the mechanical disintegration of the fractured rock mass, and from a general loss of rock shear strength through chemical alteration and replacement. The lateral extent of this deeper weathered slope mantle is determined by the distribution of the spillitic bedrock underlying the site. As well, the deeper seated nature of the slope movement at the study area is believed to be attributable to the greater depth of weathering. The postulated failure mechanisms are discussed in section 2.6.

2.44 Hydrogeology

A number of site investigations of landslides in colluvial slopes (for example, D'Appolonia et al, 1967), have shown that ground water within the colluvium is perched. This is due to the sliding surface and the underlying bedrock acting as a suitable hydraulic discontinuity to downward ground water flow. As a result of the ground water being perched, landslide material in colluvial slopes are particularly susceptible to increases in piezometric pressure (and hence slope movements), following rainfall.

At the study area, site conditions similar to those described by D'Appolonia et al are believed to exist (a non plastic, relatively permeable colluvium overlying a relatively impermeable bedrock). Therefore, there is a possibility that ground water within the Ethelton Slip is perched, and piezometers should be installed to determine this. Ground water in a sliding mass, perched above the failure surface, is unquestionably one of the principal mechanisms triggering landslide movements.

At one locality within the study area, water seepage issuing from spillitic bedrock occurs. At this locality, towards the northwest margin of the upper landslide boundary, swampy ground covering some $30m^2$ occurs at the base of a steep, unvegetated cliff of bedrock. A ditch, unsuccessfully draining the area, falls away towards the unnamed stream bisecting the landslide.

Following rainfall, the residual surface top soil covering the site has a characteristic soggy nature. Trace amounts of the swelling clay mineral chlorite, as well as a moderately high plasticity index of the clay size particles, probably accounts for this water retention.

An unnamed stream flowing from the head of the site to the toe, bisects the study area. The stream originates at the crest of the slope above the site, and has a flow capacity of approximately 0.1 cubic meters per second. At a point in the stream, midway between the county road and the railway, the flow capacity has been seen on occasions to drop to almost half that of normal. Infiltration of water into the landslide through high permeability zones at this locality is presumed to be taking place.

2.5 LANDSLIDE CLASSIFICATION

Classification of the Ethelton Slip is likely to prove complex. Skempton and Hutchinson (1969) describe slides in colluvium "in areas where severe melt-water erosion during the retreat of the Pleistocene ice sheets have been followed by some 10,000 years of virtually free degradation of over-steepened clayey or shaley slopes. A prerequisite of such landslides is a period of strong erosion, to produce the oversteepened slope, followed by a long period of little or no erosion during which colluvium can accumulate at the foot of the slope in step with weathering and degradation." The term colluvium is used in the restricted sense of the word, in that the material has been derived from its site of origin by gravity, and that the material involved in movement is weathered to the extent that relic structural features of the insitu bedrock are no longer present. Movements of the partly translational, partly rotational, types may be the mode of mass movement (Fig. 5). Perched ground water within the colluvium may regulate the slope movements.

Compound slides typically develop in slopes where a weathered soil mantle overlies unweathered bedrock. The discontinuity between regolith and rock prevents the development of a simple rotational slide. At some stage in the development of the slip, movement passes from an initial rotational element into a translational component. In general, the smaller the depth to the discontinuity, the greater the translational component will be. Material within the landslide is distorted and broken following severe shearing during movements. Rotational components in the movement may occur along several curved slide surfaces which daylight at the surface as tensional cracks. A number of slide surfaces may develop progressively at successively higher levels in the slope. The slide surfaces often coalesce at depth in the slide to produce a continuous, planar slide surface along which translational movement occurs (Fig. 5).

Skempton and Hutchinson describe a third type of movement, slump earthflows. These occupy a position transitional between rotational slides and earthflows. Rotational movement is usually dominant. Typically, the

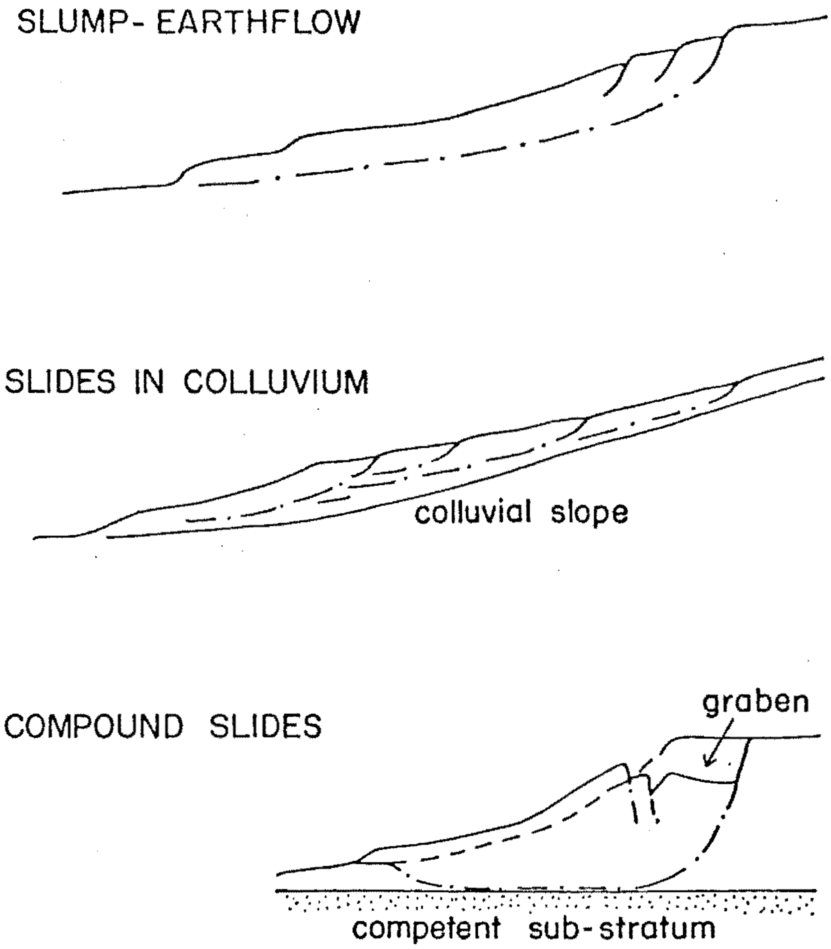


FIG. 5. LANDSLIDE CLASSIFICATION (after Skempton and Hutchinson, 1969)

foot of the failed slope is heaved; material within the toe, considerably broken by over-riding, is progressively softened until such time that a slow forming earthflow may develope (Fig. 5).

The Ethelton Slip is believed to show evidence of a combination of a slide in colluvium, a compound slide, and a slump-earthflow. In common with slides in colluvium, the slide mass involved in the movement comprises a true colluvium in the restricted sense of the meaning. The processes of weathering of the insitu bedrock to form the slide material were discussed in section 2.43. As with compound slides, a bedrock discontinuity is known to underlie the slide mass. However, the shape of the slide surface, and whether both rotational and translational movements have occurred, are unknown. Whether the shear surface is actually at the bedrock-colluvium interface, or slightly above (as happens occasionally), is also unknown. In common with slump-earthflows, the toe of the failed slope is heaved, or bulged, while the head is sunken. These are characteristics of a landslide in which principally rotational movements have occurred.

2.6 POSTULATED FAILURE MECHANISM

As landslide remedial measures must alleviate the mechanisms controlling earth movements, the likely failure mechanism of a landslide must be postulated before remedial measures are attempted.

Although insufficient site investigations have been executed, the following description is an attempt to postulate the failure mechanism of the Ethelton Slip:

(a) Rapid downcutting and undercutting by the Hurunui River through the Ethelton Gorge during, principally, the last 10-14,000 years produced relief up to 300-350m and oversteepened slopes with angles typically 30-45^{o}. This period of downcutting followed the release of considerable melt-water flows at the termination of New Zealand's last stadial period.

(b) Following downcutting and oversteepening, sets of stress-relief-induced fractures and joints developed in the near-surface Torlesse bedrock. Much of the rock was probably fractured initially, as a result of ongoing tectonism throughout the Canterbury region generally.

(c) Oversteepened slopes outside of the study area experienced disintegration of the near-surface fractured, sedimentary bedrock, producing a 2-3m thick veneer of weathered slope mantle approximately paralleling the ground surface. Landsliding within the slope mantle is believed to have resulted from overstressing within weathered material immediately above the valley floor due to the undercutting action of the Hurunui Rover. As overstressing of the material continued, the peak shear strength of the weathered mantle is believed to have been exceeded causing the failure of the slope toe. As failure of the toe of the slope commenced, overstressing of the weathered mantle was propagated to material higher in the slope. Failure of the remaining sections of the slope therefore resulted as the peak shear strength of the weathered mantle became exceeded at progressively higher levels in the slope. Longitudinal convexo-concave slope profiles indicate periods of instability within slopes outside the study area, though these slopes have now attained equilibrium. Landsliding is thought to have taken place by numerous surficial, translational-type debris flows and slides.

(d) At the site of the Ethelton Slip, bedrock consists principally of a basic volcanic lithology. Mass wasting of the slope results from the disintegration of the rock mass due to sets of stress-relief-induced fractures in the near-surface bedrock, and from a general loss of rock shear strength as a result of chemical alteration and weathering. Differential weathering between Torlesse sedimentary and volcanic lithologies has therefore resulted in weathering at the site up to possibly ten times the depth of weathering in bedrock slopes above the rest of the Ethelton Gorge.

(e) At the study area, the mechanism of mass movement was similar to that occurring in slopes above the rest of the gorge, involving a progressive overstressing of the weathered mantle at successively higher levels in the slope. At the onset of mass movement, translational-type debris slides and flows, similar to landsliding outside of the site, is believed to have occurred. However, as the depth of weathering, and thickness of colluvium, increased, a number of the small slide surfaces are believed to have coalesced at depth forming a single shear surface coincident with the colluvium-bedrock interface. It is likely that as the shear displacements increased, the shear surface became continuous from the valley floor, to the upper boundary of the site. The physiography of the landslide surface (sunken head, heaved toe) suggests that as the depth of colluvium increased, movement at the site became more rotational in nature.

(f) With increasing shear displacements, the shear surface is likely to have become highly slickensided and clay-gauge filled; as shear strains continued, clay particle-reorientation at the slide surface would occur. Thus, the shear strength mobilized at the sliding surface would approach, or be at, the residual value. As well, the failure surface shear strength will be considerably below that of the overlying intact colluvium.

(g) Ground water within the colluvium is believed to be perched above a relatively impermeable slide surface and underlying bedrock. Fluctuations in the hydrostatic level above the failure zone are therefore likely to control displacements in the colluvial slope.

2.7 FIELD INVESTIGATIONS

2.71 Rainfall Monitoring

A Marguis 600 Series rainfall gauge was installed at track level 500m southeast of the site. The gauge was installed in August, 1976 and is read daily by the district Railways Inspecting Ganger. A New Zealand Meteorological Service recording station number, H23913 (Ethelton) has been assigned to the gauge.

In addition, Mr. R. E. McFadden, "The Acheron", has kindly been recording rainfall since June, 1976 from a recording station located one kilometre south southeast of the site.

A Meteorological Service recording station, H23811 (Lowry Hills Station) has been recording rainfall from an area 10 kilometres to the north since 1947 (see Fig. 3).

Table I summarises rainfall data for all recording stations. During the period of this study, rainfall recorded at the site was approximately equivalent to the district mean for the months of August, October and November, 1976. The months of December, 1976 and January, 1977 experienced rainfall heavier than the district mean, while February, 1977 was slightly drier than normal. No months experienced exceptionally heavy rainfall.

2.72 Landslide Performance

Eight surface survey markers were installed during March, 1976. Their locations are shown on the Site Plan. Survey markers 1, 6 and 8 were installed outside the slip. All other markers are in potential zones of movement.

Survey markers were constructed by welding a flat, square iron plate to a 2-2.5m length of railway line. At their respective locations, holes were dug and survey markers placed to a depth approximately half their length. The welded plate at the top of each marker was levelled, and a grout composed of cement, sand and gravel was packed in the hole. A circular anodised alluminium instrument plate was finally screwed into position above the iron plate. A circular hole through both iron and aluminium plates allow survey instruments to be bolted securely onto each survey station (Fig. 6).

An initial survey of all stations was performed by the 1976 3rd Professional survey class, School of Engineering, University of Canterbury, during 10 and 11 April, 1976 under the instruction of Mr. D. R. Gordon, Senior Lecturer, School of Engineering.

TABLE I

MONTHLY RAINFALL FOR RECORDING STATION LOWRY HILLS (1947-76), AND WEEKLY RAINFALL FOR STATIONS ETHELTON (AUGUST 1976-FEBRUARY 1977) AND "THE ACHERON" (JUNE 1976-JANUARY 1977)

Figures in millimetres of rainfall.

	LOWRY HILLS				ETHELTON					"THE ACHERON"				
	1	2	3	4	WK1	WK2	WK3	WK4	Tot	WK1	WK2	WK3	WK4	Tot
January	69	204	9	57	12	20	79	17	128	5	21	85	0	106
February	59	146	16	142	7	16	7	1	31					
March	69	207	7	29										
April	87	268	21	76										
May	89	311	11	38										
June	67	233	14	47						11	0	22	20	53
July	83	346	9	97						9	26	75	3	113
August	82	249	10	56	18	35	13	14	880	21	34	13	34	102
September	53	271	9	85	60	54	6	10	130	61	57	7	12	137
October	70	175	12	70	5	36	27	15	83	4	38	32	17	91
November	61	209	5	57	9	12	1	23	45	13	15	0	48	76
December	68	213	10	103	21	15	44	14	94	19	17	38	20	94
Total	857	1227	405	857										

1 = mean (1947-75), 2 = high (1947-75), 3 = low (1947-75), 4 = 1976 figures.

This initial survey allowed a ground control co-ordinate system to be constructed to enable future slip movements to be observed. A Wild T3 theodolite ($\pm$ 0.2 secs) was employed to observe bearings and vertical angles, while a Wild Distomat DI3 and a Kern Geodimeter recorded slope distances.

Slope distances and bearings were recorded in the following manner:

from Station No.	to Station No.
1	all other stations
2	1, 3, 4, 5
3	1, 2, 4, 5
4	1, 2, 3, 5
5	1, 2, 3, 4
6	1
7	1
8	1

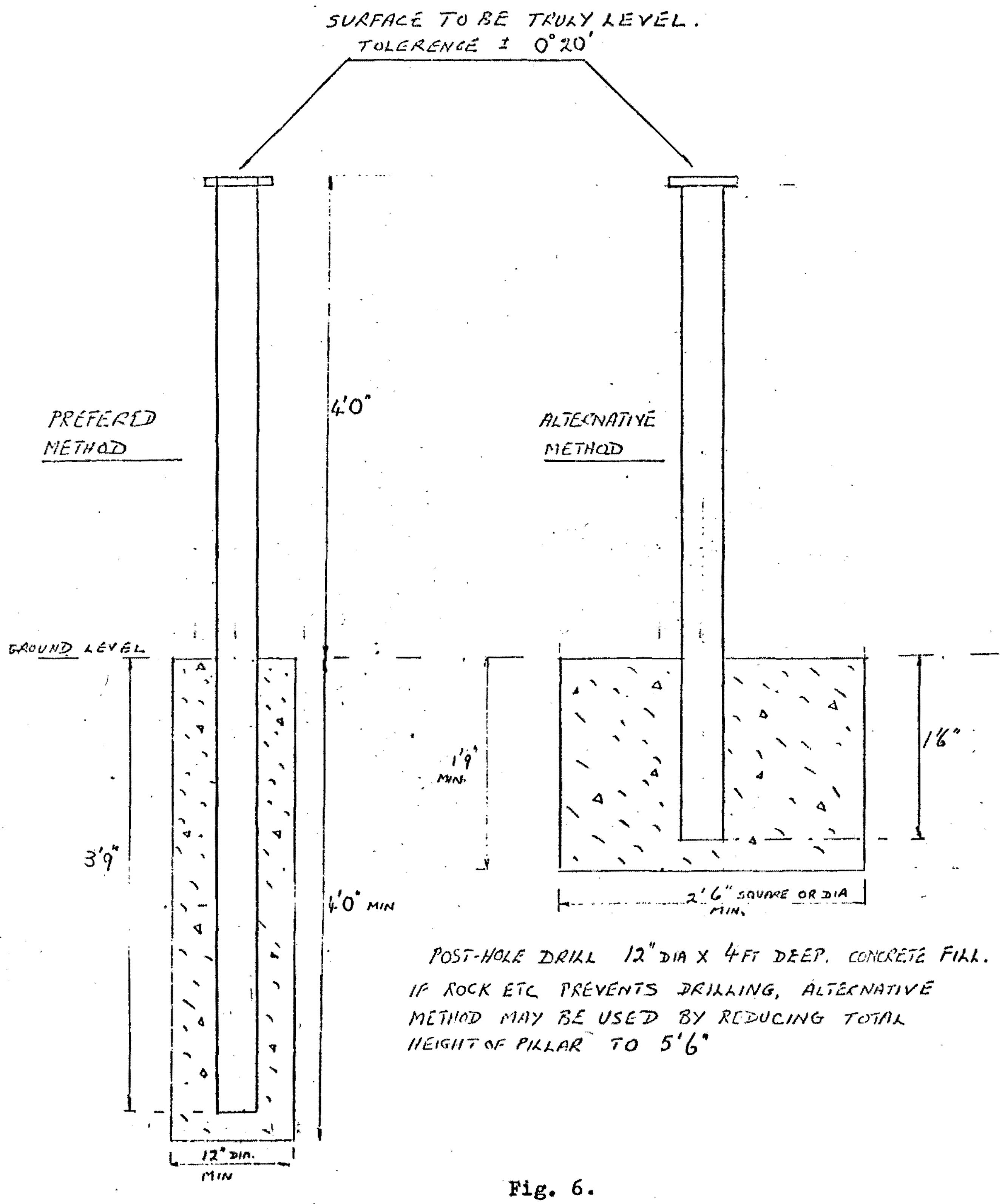
Those stations located in more than one way by both bearing and distance constitute a redundant system in which there is a check on the accuracy of measurements. It is anticipated that at least one additional survey marker will be installed so that all stations located in the slip will constitute a redundant system in the future.

A computer programme, written by the School of Engineering, allowed all raw data from the initial survey to be reduced by a least squares process. The method enables random errors and measurement inconsistencies to be accounted for in the calculations.

The results of calculations by the School of Engineering enabled all eight surface survey markers to be

CONTROL STATION.

INSTALLATION



POST-HOLE DRILL 12" DIA X 4FT DEEP. CONCRETE FILL.
IF ROCK ETC PREVENTS DRILLING, ALTERNATIVE METHOD MAY BE USED BY REDUCING TOTAL HEIGHT OF PILLAR TO 5'6"

Fig. 6.

assigned reduced levels and coordinates; coordinates were tied in to the New Zealand Coordinate System. In addition, the bearings and horizontal distances from Station No. 1 to all other stations were calculated.

Re-surveys of surface markers have been executed by the writer since their installation at approximately two monthly intervals. A Zeiss 010 theodolite measured bearings and a Wild Distomat DI3 recorded slope distances. All observations were taken from control survey Station No. 1. The following survey method was used:

from Station No.	to Station No.
1	2, 3, 4, 5, 7 bearing, vertical angle, slope distance.
1	6, 7 bearing, vertical angle.

Four theodolite measurements were observed for each station, two on both right and left face, with 90° difference between observations on the same face. Up to ten slope distance measurements were recorded.

Slope distance measurements to survey stations 6 and 8 were not undertaken as these lie outside the slip in presumably stable ground (station 8 is also outside the Distomat range).

The writer is aware that the method used during the re-surveys, in which all observations were recorded from a single control station, does not constitute a redundant system. However, the method used does not mean a reduction in the accuracy of the results; rather, measurement inconsistencies and random errors will not be so easily detected as in a redundant system.

The accuracy of the survey method as a whole has been calculated on the following basis:

(a) At the 67% (one sigma) confidence level the Distomat DI3 has a 5mm RMS (root mean square). Over the range worked at Ethelton, that of the Zeiss (010) theodolite is given as 1.5-2mm RMS. Therefore, at the 67% confidence level, the accuracy of the method is given as:

$$\text{SQRT } (5^2 + 2^2) = 5.4\text{mm}$$

(b) Similarly, at the 95% (two sigma) and 99.9% (three sigma) confidence level, the accuracy of the method is:

$$95\% : \text{SQRT } (10^2 \times 4^2) = 10.8\text{mm}$$

$$99.9\% : \text{SQRT } (15^2 + 6^2) = 16.2\text{mm}$$

(c) Accepting the 99.9% confidence level as most suitable, as well as allowing for operator inexperience, a 20mm margin of error has been applied. Therefore, any movement recorded on surface markers within the quoted accuracy is automatically disregarded.

In addition, the method used during the re-surveys contains no absolute check on the stability of the three stations, 1, 6 and 8, outside the slip area; such a check will only be undertaken during each subsequent annual survey by the School of Engineering. However, as the bearing difference between fixed stations 6 and 8 from control station 1 remained constant throughout the re-surveys, the stability of the three fixed markers has not been questioned.

A Fortran computer programme, compiled by Mr. T.C. Clisby, assistant engineer, N.Z. Railways, was written for use on the University of Canterbury's Burroughs B6718 computer. The programme has the ability to reduce all raw data collected during the re-surveys.

The programme calculates the average bearing, vertical angle and horizontal distance from station No. 1 to all other stations. New coordinates and reduced levels for all stations, except control marker No. 1, are calculated, and differences with those of the initial survey are noted. Finally, rates of movement, magnitude of movement and direction of movement are analysed.

Results of the four re-surveys are given in Appendix I.

The following conclusions concerning precise surveying of surface markers have been drawn:

(a) With the exception of survey marker No. 4, all stations located in the landslide have shown no significant movement since the initial survey of 11.4.76 to the final survey of 3.2.77. However, it should not be ruled out that movements are not occurring on all stations except marker No. 4. If significant displacements are occurring on these markers, it is obvious a longer survey period will be needed to establish trends.

(b) Station No. 4 has experienced a total decrease in horizontal distance with respect to fixed station No. 1 of 11.9cm and a lowering of reduced level by 5.0cm (Fig. 7). Resolved into a three-dimensional magnitude of movement, this results in a total displacement of 13.1cm during the 9.7 month survey period of this study, or an inferred 16.2cm annual displacement.

(c) The direction of movement of station No. 4 is consistent with a downward and outward displacement towards the Hurunui River at the toe of the slope (Fig. 8).

(d) Figure 9 summarises an attempted correlation of rainfall with rates of movement for marker No. 4. Fortuitiously or otherwise, the highest and lowest rates of movement correlate, respectively, with the highest and lowest rates of precipitation received at the site. However, it is the writer's opinion that the length of time of this study is too short for a correlation of rainfall with movement to be definitely established; a longer period of surveying will be needed to effect a correlation, if one exists.

(e) Movement on survey marker No. 4 is in agreement with field studies which suggest that displacements at the site are presently confined to a relatively small area in the lower (foot) region of the landslide.

(f) Insufficient time yet exists to establish a minimum rate of precipitation above which rates of movement within the active zone can be expected to accelerate. However, Figure 9 does tentatively infer a value of 100mm per month of rainfall received at the site above which rates of movement can be expected to increase.

2.73 Seismic Refraction Traverse

Two seismic refraction traverses were undertaken. The traverse locations, shown on the Site Plan, were sited to record a cross-section and a longitudinal section through the site.

A dual-channel seismograph (Bison 1501) utilizing a single geophone without a filter gate was used. A 20kg hand hammer impacted on the ground surface provided the

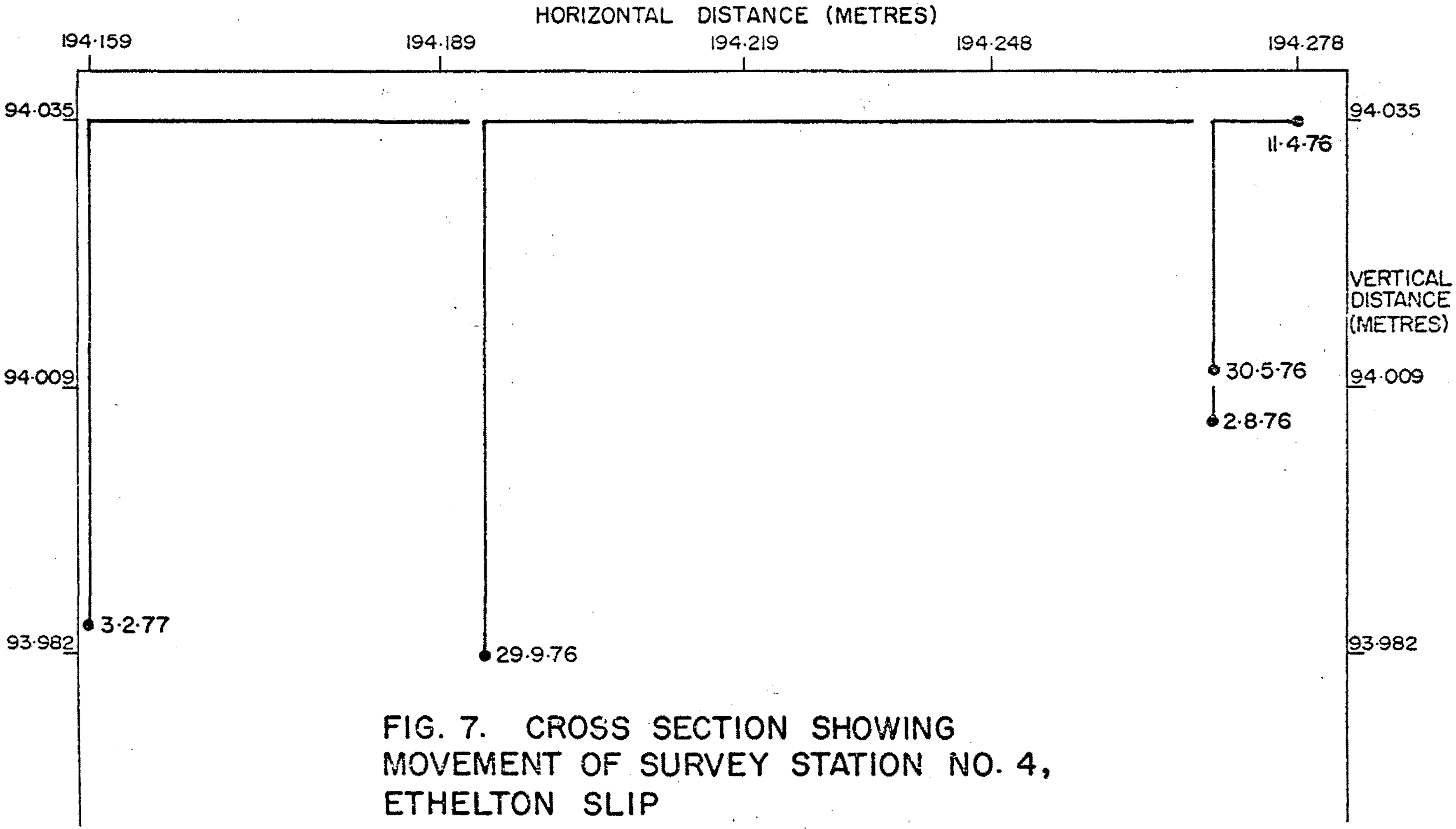


FIG. 7. CROSS SECTION SHOWING MOVEMENT OF SURVEY STATION NO. 4, ETHELTON SLIP

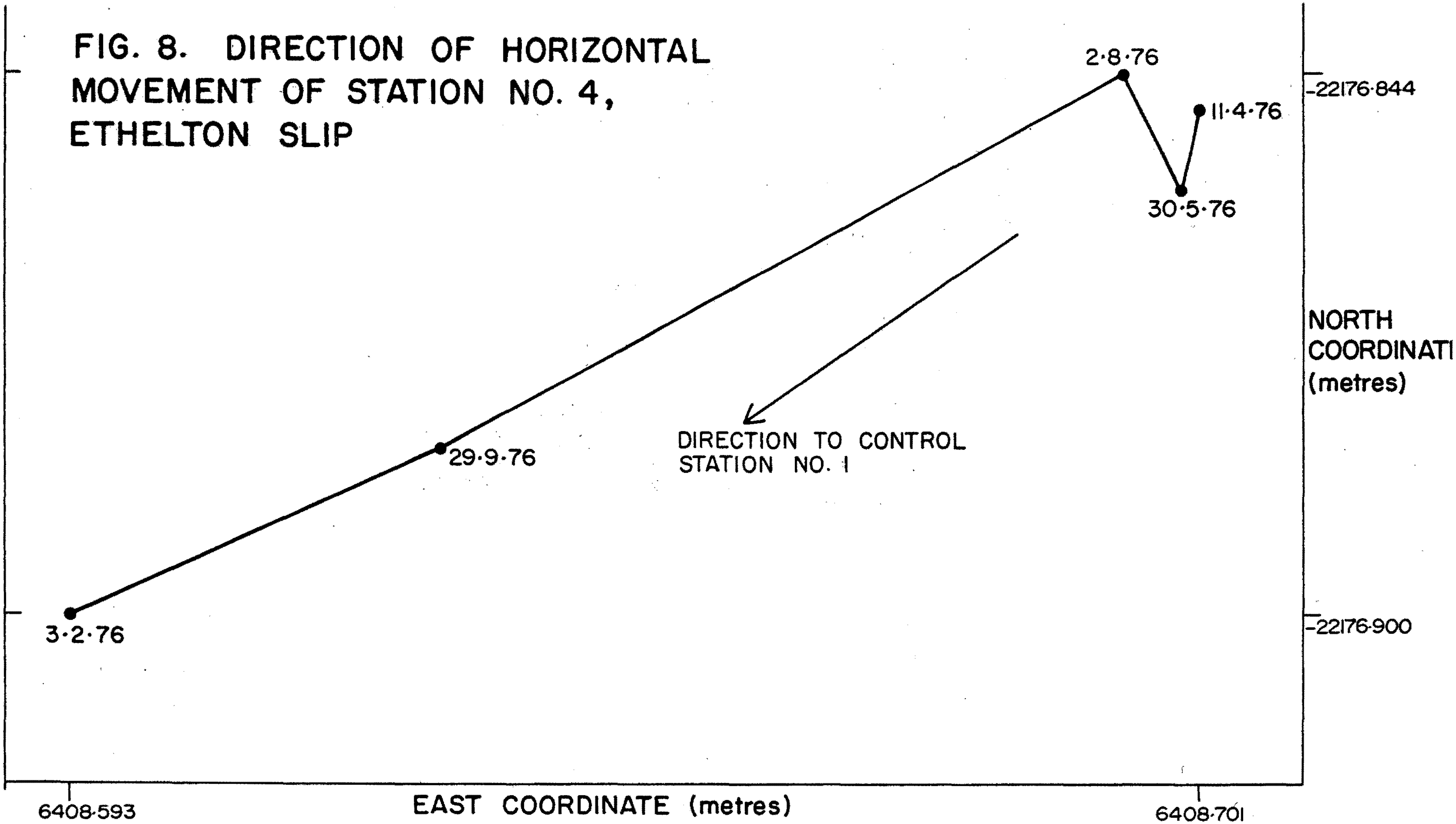


FIG. 8. DIRECTION OF HORIZONTAL MOVEMENT OF STATION NO. 4, ETHELTON SLIP

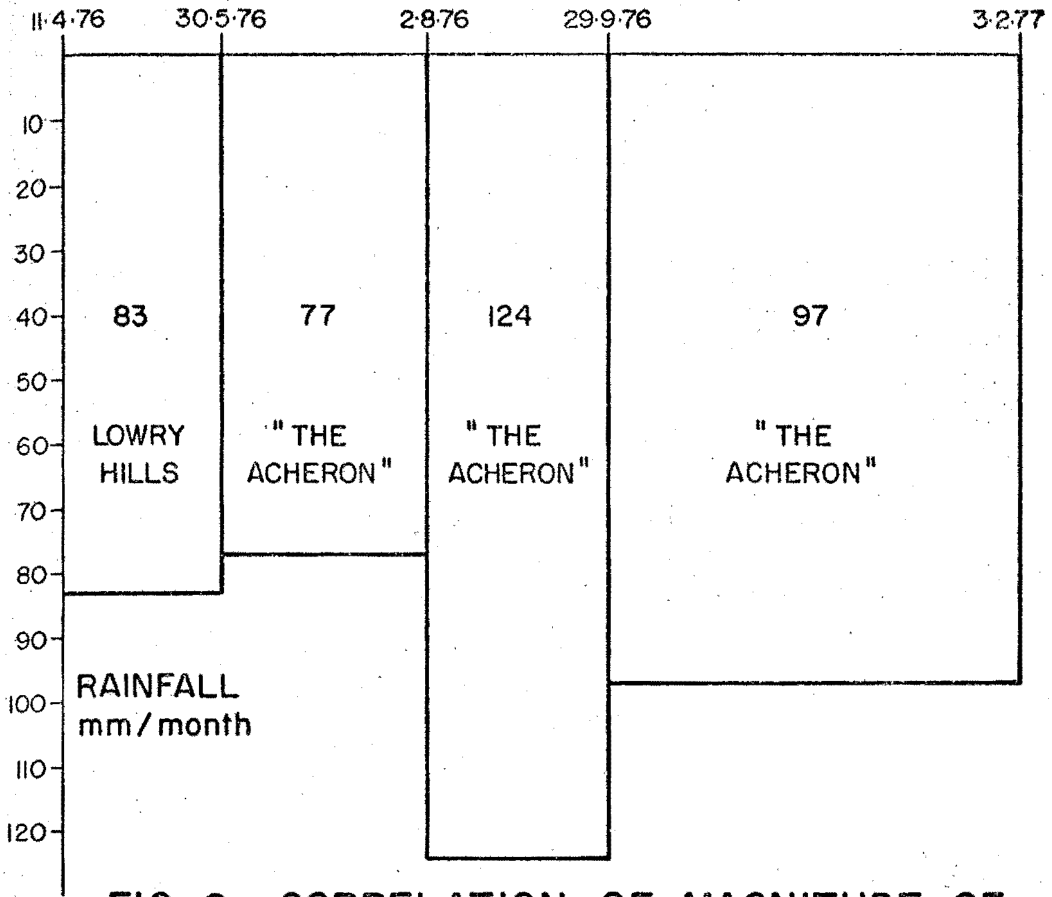
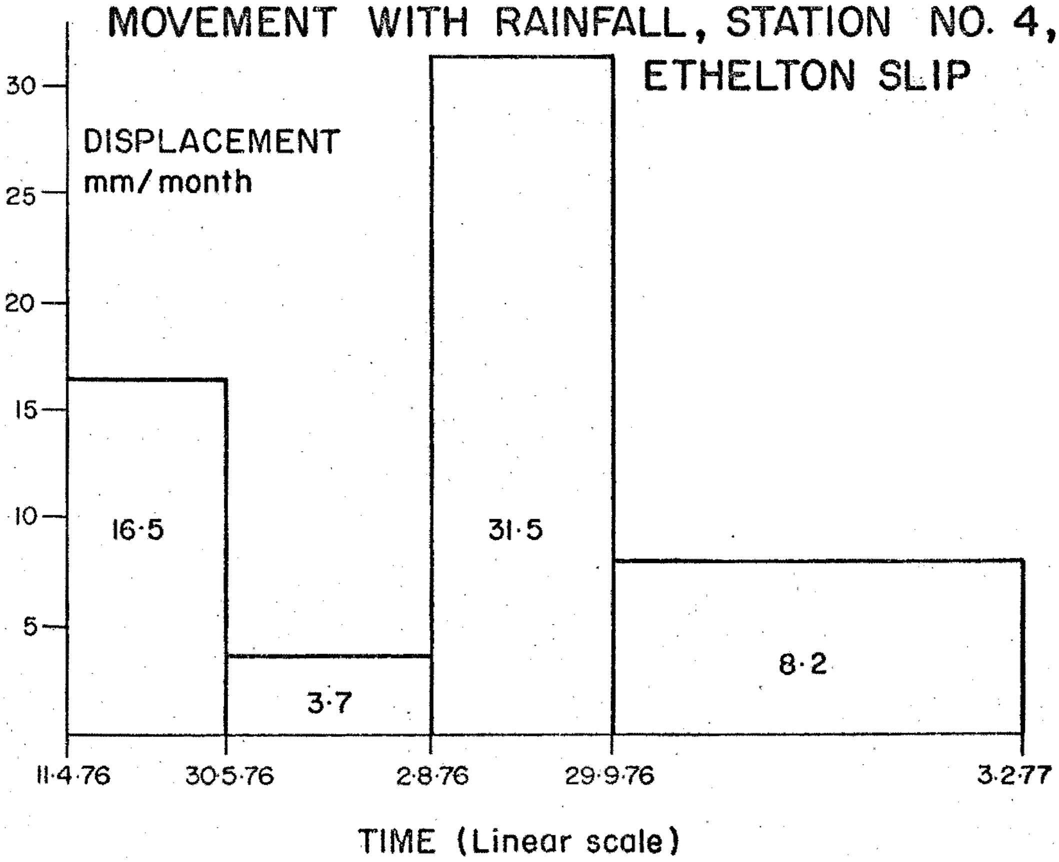


FIG. 9. CORRELATION OF MAGNITUDE OF MOVEMENT WITH RAINFALL, STATION NO. 4, ETHELTON SLIP



energy source.

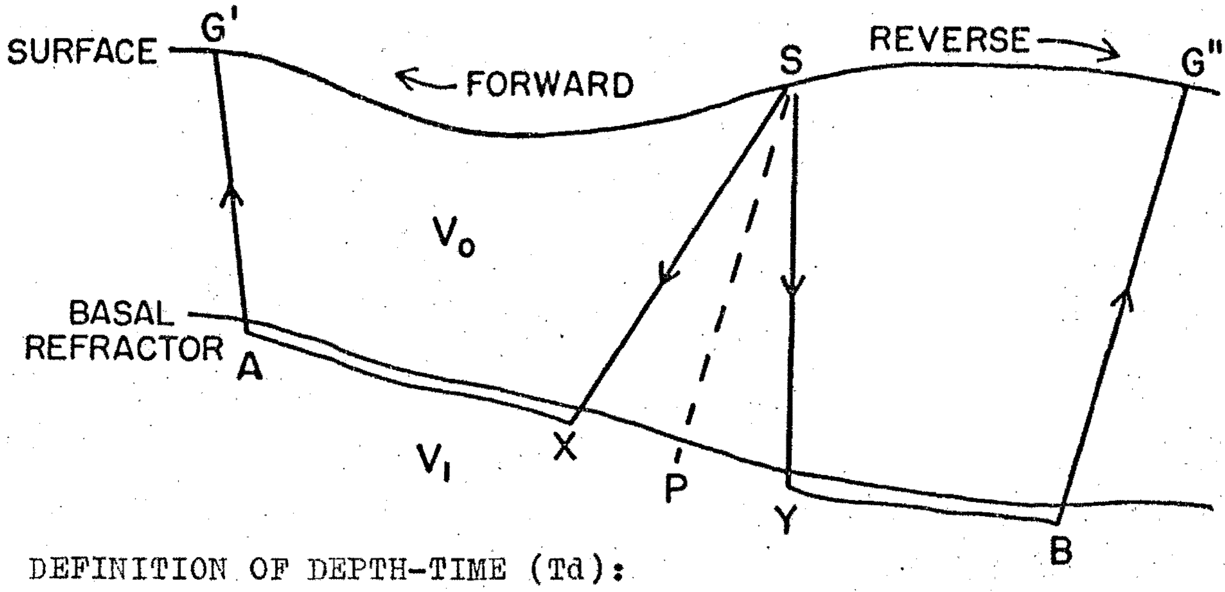
Traverses were surveyed by theodolite and staff and their locations and reduced levels on the landslide surface tied in with the earlier topographical survey undertaken for the preparation of the base map (section 2.2).

The seismic refraction survey undertaken at the site is based on the reciprocal method of Hawkins (1961). The technique is particularly suited to recording absolute depths to shallow single or two-layered refractors. Large velocity contrasts between overbuden and bedrock allow the method to be used widely as a site investigation on engineering constructions.

The reciprocal method of determining depths to refractors is based on the calculation of the "depth-time". The depth-time to a refractor is equal to the time taken by the critical ray path to travel from the refractor to the ground surface, minus the time required to travel the projection of the ray on the plane of the refractor at the velocity of the refractor (Fig. 10).

Results of the two seismic refraction traverses are shown in Tables 2a and 2b and Figures 11a, 11b and 12. Along traverse No. 1 a single layer with a seismic velocity of V_1 = 620m/sec. overlying a basal refractor of seismic velocity V_2 = 1780m/sec. was calculated. The single layer model has been interpreted as a colluvium of weathered slope mantle overlying a basal bedrock refractor. Thicknesses of interpreted colluvium along traverse No. 1 are fairly consistent, varying between 4-5m.

A single layer model has also been calculated along traverse No. 2. The layer has a seismic velocity of



DEFINITION OF DEPTH-TIME (Td):

$$Td = \frac{SX}{V_o} - \frac{XP}{V_1} \qquad \text{or, } Td = \frac{SY}{V_o} - \frac{YP}{V_1}$$

CALCULATION OF DEPTH-TIME:

Forward time (Ft): $\frac{SX}{V_o} + \frac{AX}{V_1} + \frac{AG'}{V_o}$

Reverse time (Rt): $\frac{SY}{V_o} + \frac{BY}{V_1} + \frac{BG''}{V_o}$

$$Ft + Rt = \frac{SX + SY}{V_o} + \frac{AX + BY}{V_1} + \frac{AG' + BG''}{V_o}$$

where: $\frac{AX + BY}{V_1} = \frac{AB}{V_1} - \frac{XP}{V_1} - \frac{YP}{V_1}$

$$\therefore Ft + Rt = \frac{SX}{V_o} - \frac{XP}{V_1} + \frac{SY}{V_o} - \frac{YP}{V_1} + \frac{AB}{V_1} + \frac{AG' + BG''}{V_o}$$

$$Ft + Rt = 2Td + \frac{AB}{V_1} + \frac{AG' + BG''}{V_o}$$

where: $\frac{AB}{V_1} + \frac{AG' + BG''}{V_o}$ = Reciprocal time (Tr)

$$\therefore Td = \frac{Ft + Rt - Tr}{2}$$

FIG. 10. RECIPROCAL SEISMIC REFRACTION TECHNIQUE (based on Hawkins, 1961)

V_1 = 280m/sec. and overlies a basal refractor with seismic velocity of V_2 = 1210m/sec. Colluvium overlying bedrock is again interpreted for traverse No. 2. The colluvial thickness along this run ranges between 1.5-2.5m.

The range in seismic velocity within the colluvium between V_1 = 620m/sec. and V_1 = 280m/sec. along the two traverses is difficult to explain. A seismic velocity of 620m/sec. appears to be somewhat high for a material having soil-like properties. However, as the line of traverse No. 1 is located along the county road, recompacted fill underlying the road may possibly explain the discrepancy.

As occasionally arises in seismic refraction studies, seismic boundaries may not be coincident with actual geological boundaries. A correct interpretation of the seismic studies undertaken at the site will therefore only be obtained after subsurface investigations have been executed.

2.8 LABORATORY TESTING

An elementary and far from complete laboratory testing programme was undertaken on two residual soil samples derived from volcanic (spillitic) bedrock lithologies.

The tests include insitu moisture content, and Index tests on the finer than .04mm fraction. These tests were performed according to New Zealand Standard Specifications (1976). As well, X-ray diffraction analyses on the clay fraction were also performed. X-ray diffraction techniques allow the crystal structure of clay mineral particles amorphous to X-rays to be determined. As each clay mineral will have a characteristic crystal structure, the type of

TABLE 2a

RESULTS OF SEISMIC TRAVERSE No. 1

Reduced Level (m)	Traverse Distance (m)	Forward Time (m.sec)	Reverse Time (m.sec)	Depth Time (m.sec)	Corrected Forward Time (m.sec)	Corrected Reverse Time (m.sec)	F Value	Radial Depth F.dt (m)
108.1	0		30.3					
108.1	2.5	3.9	28.8	0.9				
108.0	5.0	6.4	27.7	1.6				
108.0	7.5	8.5	27.3	2.5				
108.0	10	14.9	26.5	5.3				
107.9	12.5	18.2	25.7	6.5	11.7	19.2	0.664	4.32
107.8	15	20.6	25.3	7.5	13.1	17.8	0.6654	4.99
107.6	17.5	20.7	23.1	6.5	14.1	16.6	0.6664	4.33
107.6	20	23	21.6	6.9	16.1	14.7	0.6676	4.61
107.5	22.5	24.3	20.2	6.8	17.5	13.4	0.6687	4.55
107.4	25	25.6	19.1	6.9	18.7	12.2	0.670	4.62
107.2	27.5	27.5	13.3	5.0				
107.1	30	29.8	8.9	3.9				
107.0	32.5	29.9	3.6	1.3				
106.9	35	31.5						

Reciprocal time = 30.9 V_1 Forward = 690 m/s

V_1 Reverse = 550 m/s V_2 = 1780 m/s

TABLE 2b

RESULTS OF SEISMIC TRAVERSE No. 2

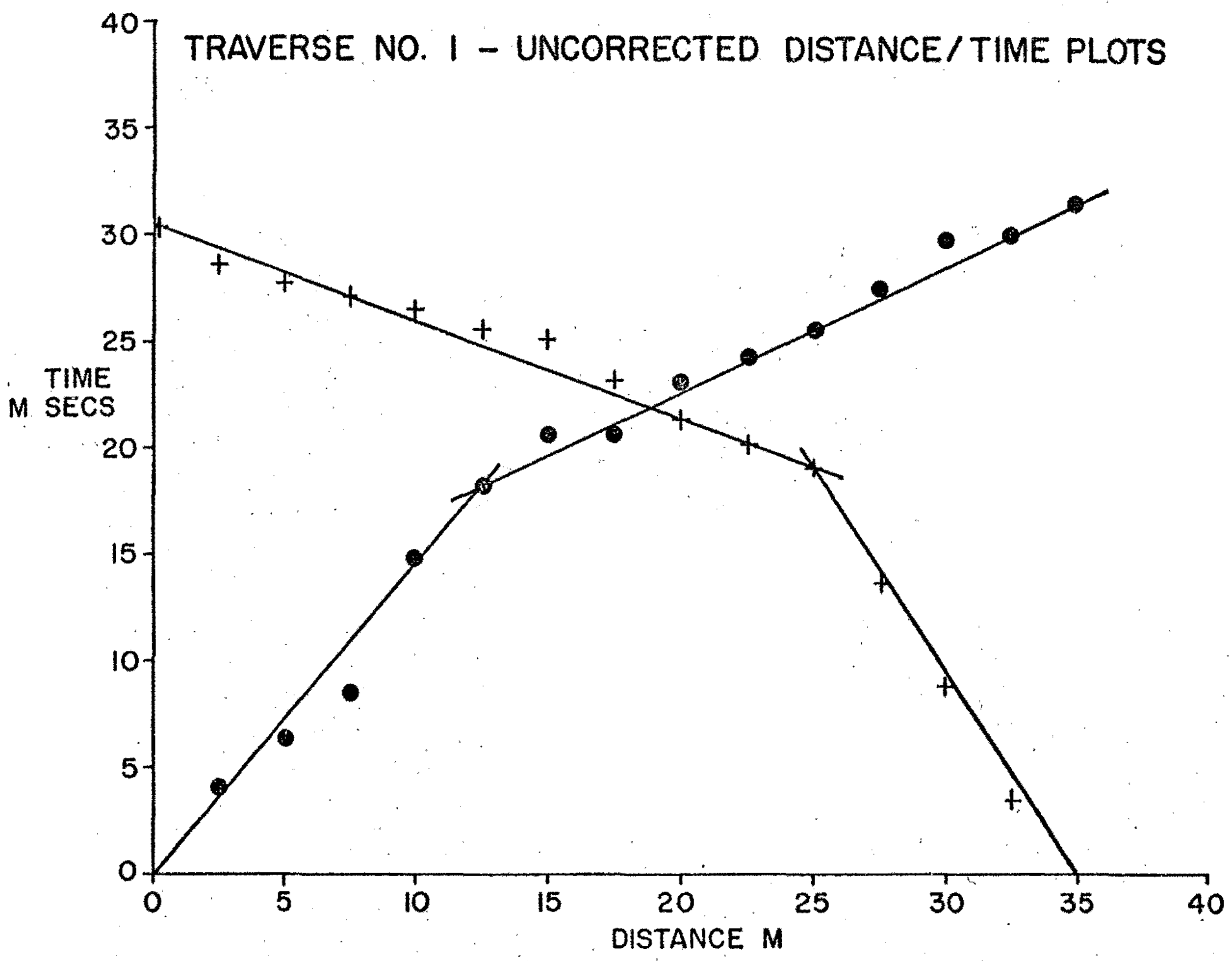
Reduced Level (m)	Traverse Distance (m)	Forward Time (m.sec)	Reverse Time (m.sec)	Depth Time (m.sec)	Corrected Forward Time (m.sec)	Corrected Reverse Time (m.sec)	F Value	Radial Depth F.dt (m)
193.3	0		35.0					
193.4	1	1.6	34.0	0.6				
193.5	2	3.2	33.4	1.1				
193.8	4	10.2	32.4	4.1				
193.7	5	12.5	31.6	4.9	7.6	26.7	0.288	1.41
193.6	6	13.6	31.3	5.3	8.3	26.0	0.288	1.53
193.5	8	15.5	30.3	5.7	9.8	24.6	0.288	1.64
193.3	10	17.7	28.8	6.1	11.6	22.7	0.288	1.76
193.2	12	21.0	28.1	7.4	13.6	20.7	0.288	2.13
193.0	15	23.1	26.4	7.6	15.5	18.8	0.288	2.19
192.8	17	26.8	24.2	8.3	18.5	15.9	0.288	2.39
192.8	18	26.8	23.8	8.1	18.7	15.7	0.288	2.33
192.4	20	28.5	21.2	7.7	20.8	13.5	0.288	2.22
192.3	22	27.6	19.3	6.3	21.3	13.0	0.288	1.81
192.1	24	30.0	18.0	6.8	23.2	11.2	0.288	1.96
192.0	25	30.5	17.0	6.6	23.9	10.4	0.288	1.90
192.0	26	31.4	16.2	6.6	24.8	9.6	0.288	1.90
191.9	28	32.2	12.0	4.9				
191.9	29	32.7	6.6	2.5				
191.8	30	33.7						

Reciprocal time = 34.4

V_1 Forward = 395 m/s

V_1 Reverse = 167 m/s

V_2 = 1210 m/s


TRAVERSE NO. I - UNCORRECTED DISTANCE/TIME PLOTS
TIME
M SECS
DISTANCE M
0
5
10
15
20
25
30
35
40

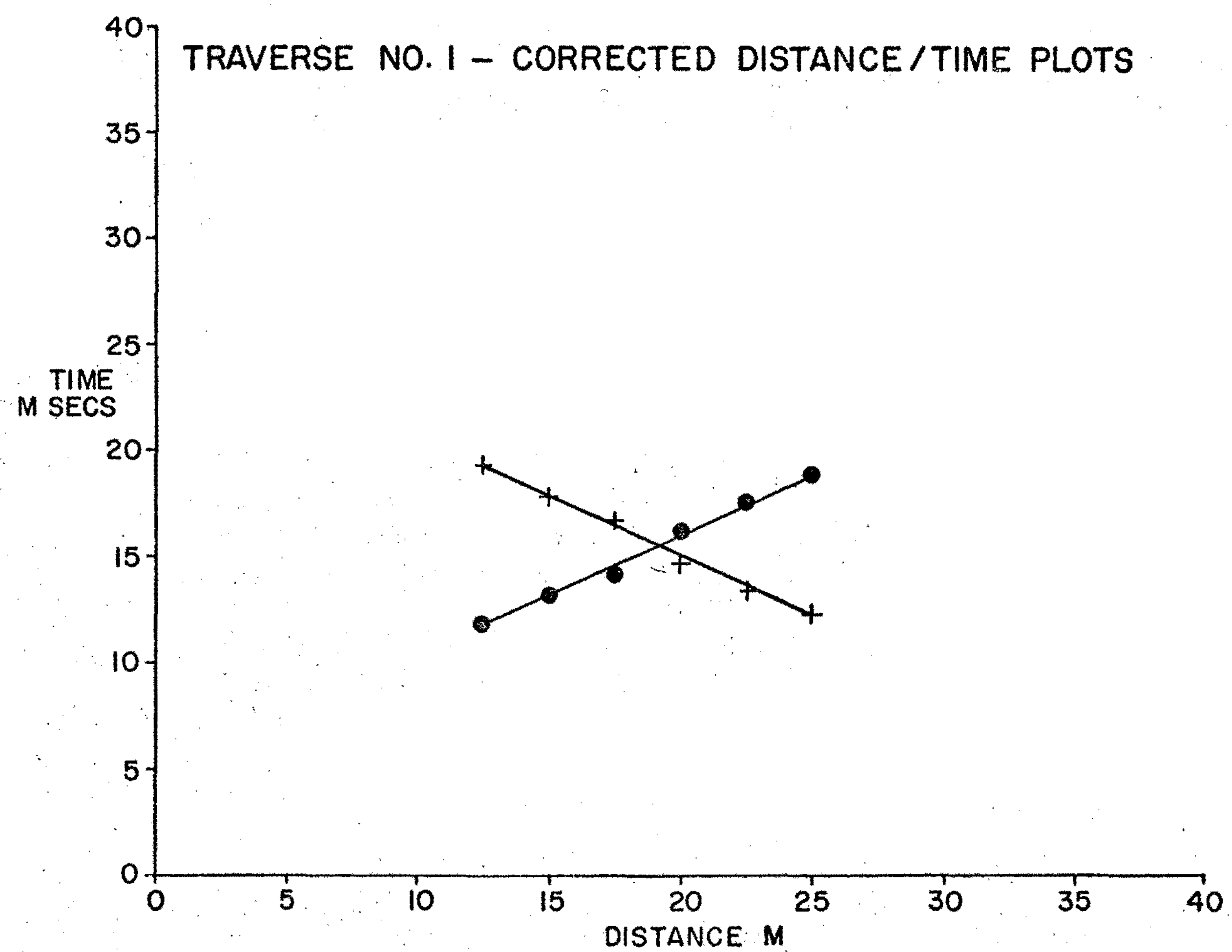
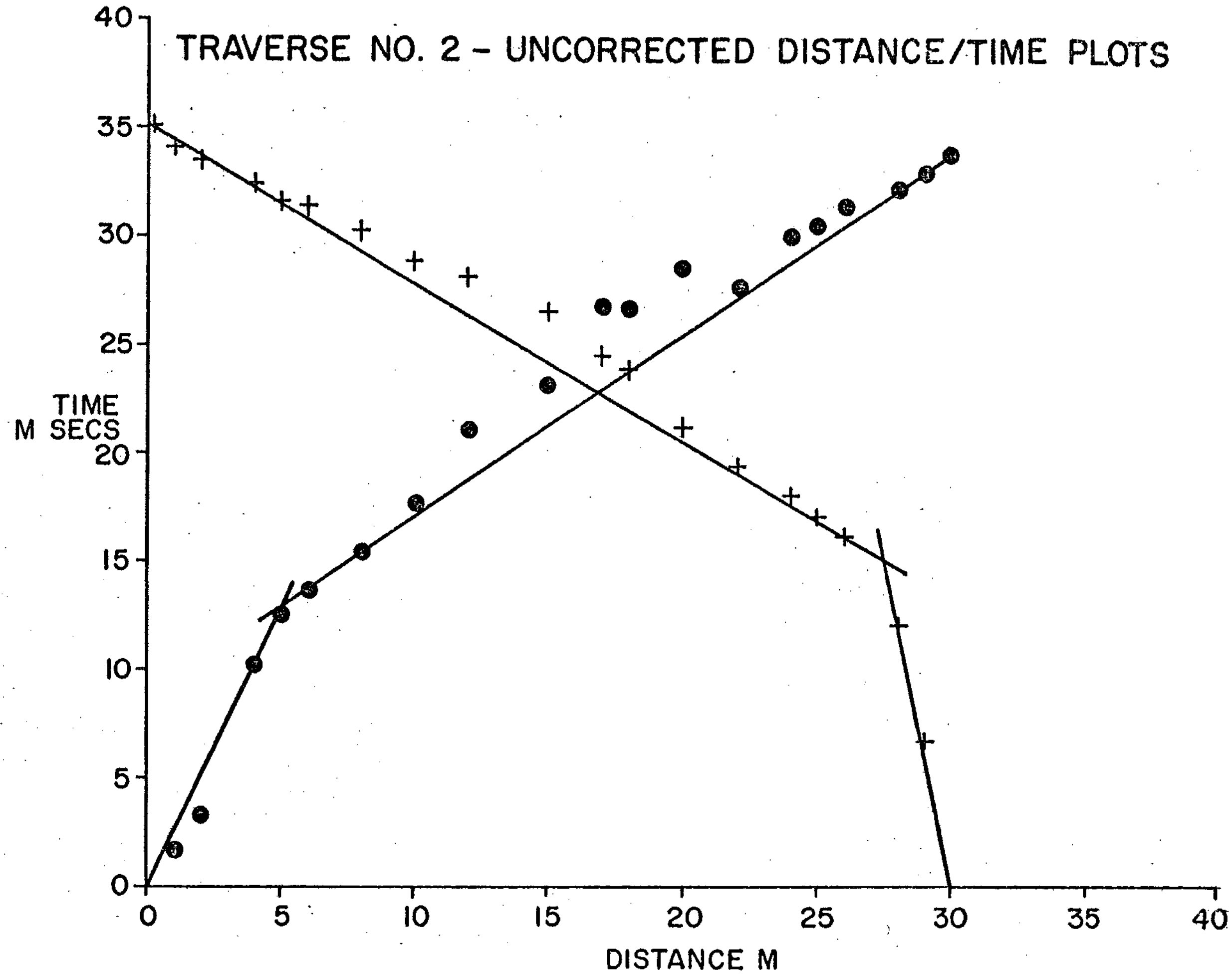

TRAVERSE NO. I - CORRECTED DISTANCE/TIME PLOTS
TIME
M SECS
DISTANCE M
0
5
10
15
20
25
30
35
40

FIG. 11a


TRAVERSE NO. 2 - UNCORRECTED DISTANCE/TIME PLOTS
TIME
M SECS
DISTANCE M
0
5
10
15
20
25
30
35
40

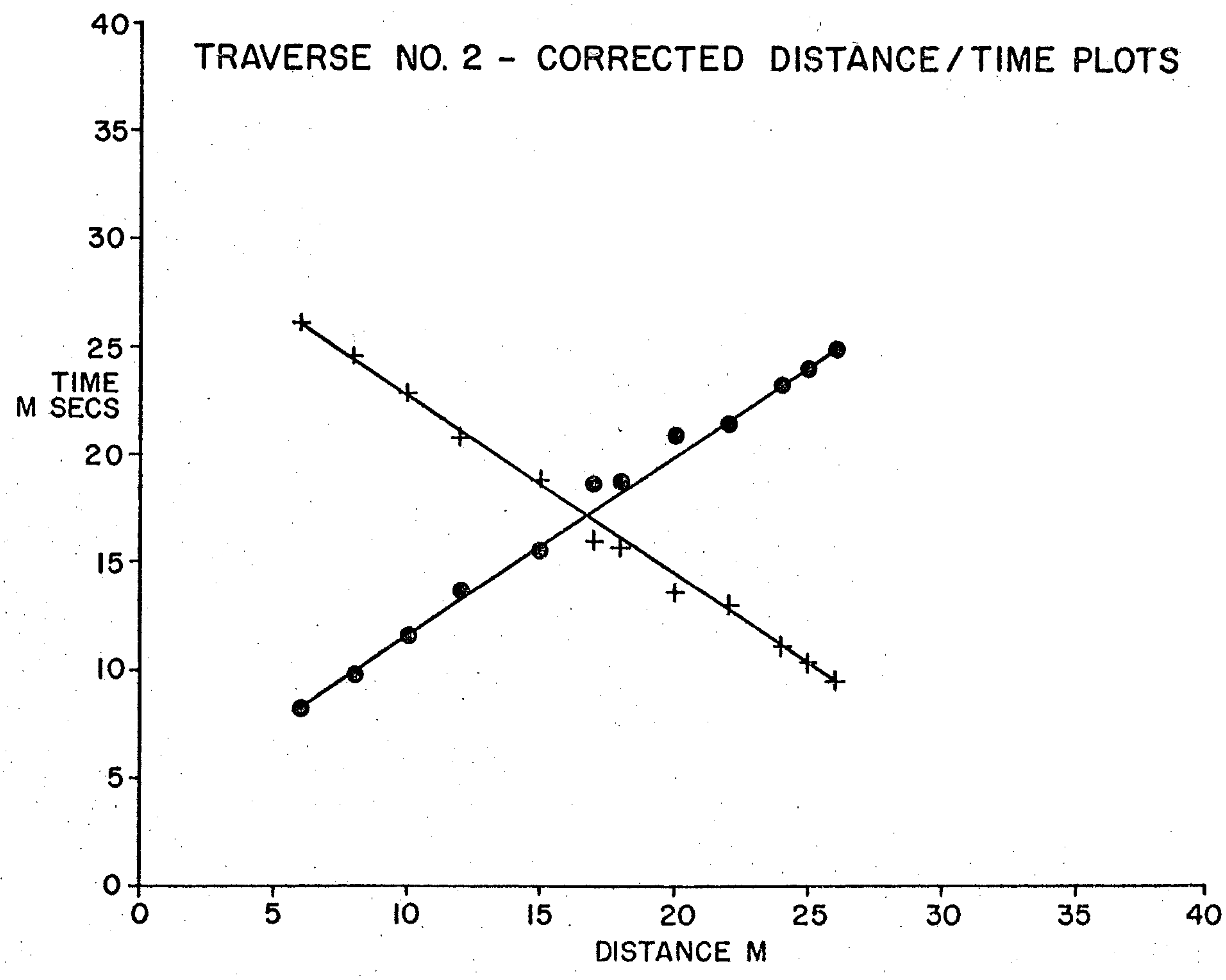
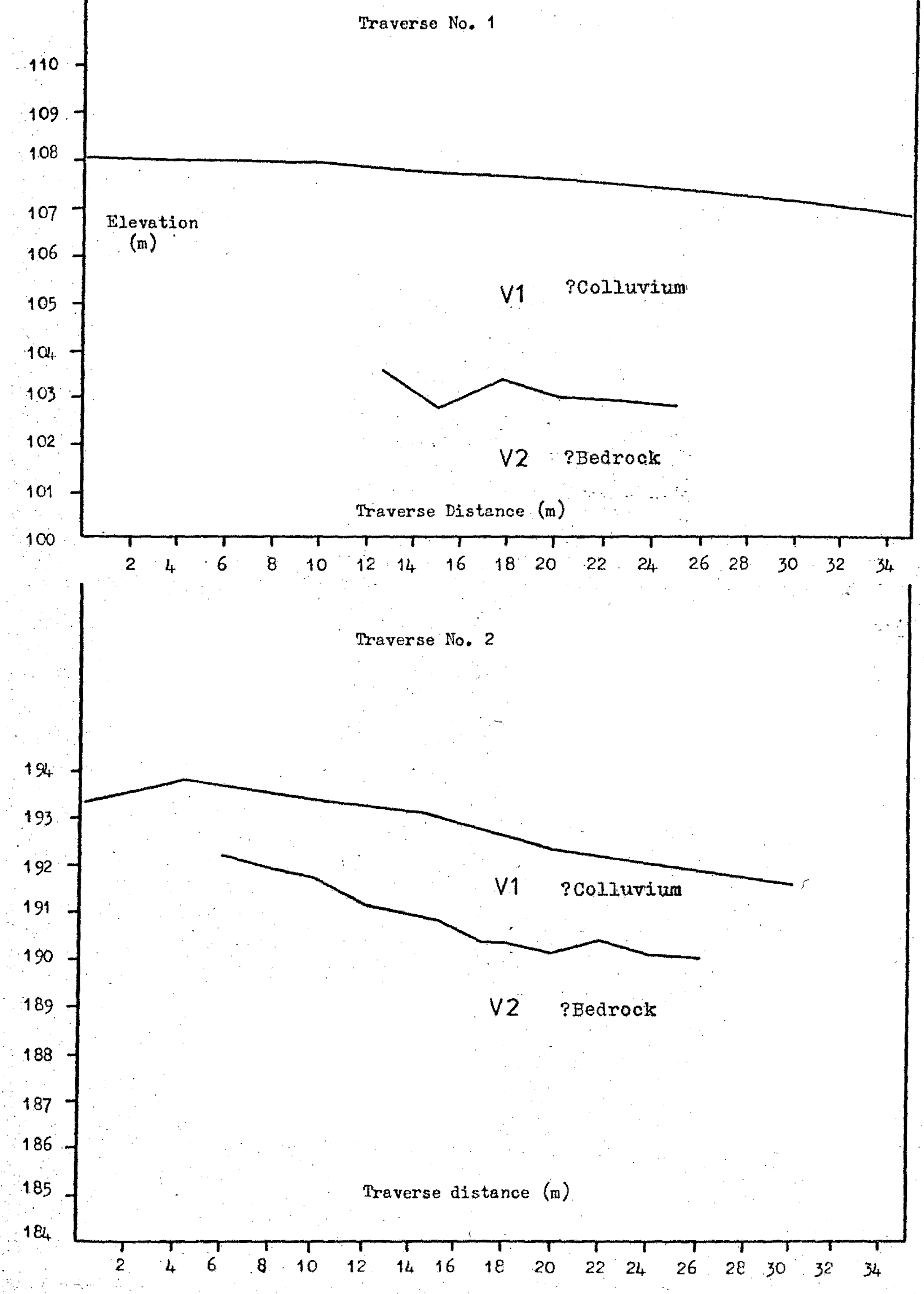

TRAVERSE NO. 2 - CORRECTED DISTANCE/TIME PLOTS
TIME
M SECS
DISTANCE M
0
5
10
15
20
25
30
35
40

FIG. 11b

Fiqure 12. Interpretative results of seismic refraction traverses.



clay mineral can therefore be identified. The technique is performed by allowing a monochromatic pencil of X-rays to pass through a powdered specimen of the clay mineral in various orientations. Where the X-rays hit the different atomic planes within the clay mineral, diffraction occurs. The characteristic spacing which each clay mineral has between atomic planes can then be measured. The spacings between the atomic planes are measured in terms of Angstroms (Å), one Angstrom being equivalent to 10^{-7}mm.

Table 3 summarises results of laboratory testing. It should be emphasised that, as the soil properties of the material comprising the shear surface are likely to be different to those of the overlying colluvial slide mass, the values of insitu moisture and Index tests presented in Table 3 are unlikely to be representative of conditions at the shear surface. Index tests indicate the colluvium is predominantly non plastic. X-rays diffraction analyses reveal the presence of a swelling chlorite clay mineral. This has been suggested in Section 2.44 as a reason for the high water retention within residual top soil material following rainfall.

TABLE 3

RESULTS OF LABORATORY TESTING ON WEATHERED ETHELTON SLIP SAMPLES

Material	In situ Moisture	Index Tests PL	LL	PI	X-ray Diffraction
Red Coloured	17.4	24.0	38.8	14.8	chlorite epidote calcite feldspar
Green Coloured	16.3	25.8	36.8	11.0	

2.9 RECOMMENDED SITE INVESTIGATIONS

At present, five surface precise surveying markers are stationed in areas of potential movement. Of these, four are positioned on or adjacent to the railway, while only one, located above the county road, is sited to record movements higher in the slope.

Additional survey markers should be installed at levels in the slip higher than the railway. These will locate further areas of instability, if any, outside the zones of activity located during field studies. As well, should stability control measures be undertaken at the site, the performance of the landslide following such action will need to be carefully monitored to determine the success or otherwise of the measures.

Recommended locations for additional survey stations are shown on the Site Plan. The more important of these include further stations in the vicinity of markers 4 and 7 in order that the extent of the present active zone may be determined. All future stations should be installed so that they may be observed from station No. 1 and at least one other marker.

Detailed site investigations in areas of slope instability usually include subsurface exploration techniques. Coring of small diameter (4-8cm) diamond drill holes is the usual method employed to obtain information on the depth of slide material, shape of the failure surface, and undisturbed soil samples for laboratory testing.

On unstable slopes where relic structural features of the slide material may be completely destroyed by mass movements, interpretation of diamond drill cores is often difficult. In these circumstances visual examination of large diameter (1m or greater) shafts through the slide mass usually provides answers to problems often unattainable by diamond drilling.

At the site of the Poro-o-Tarao Tunnel, North Island Main Trunk Railway, subsurface investigations included a series of drill holes and exploratory shafts. The interpretation of diamond drill holes, many of which experienced poor core recovery, was greatly aided by examination of large diameter shafts (Parton, 1974). As well, costs of excavating shafts were found to be compatible with rotary diamond drill holes of comparable depth.

A number of large diameter exploratory shafts would greatly aid understanding of landslide mechanisms operating at the Ethelton Slip. Calweld bucket-auger shafts of diameter 1m or more have been successfully used by the Ministry of Works and Development in New Zealand.

Excavation of exploratory shafts would furnish the following information:

(a) A visual examination of both weathered slide material and underlying insitu bedrock could be made.

(b) Depths of slide material determined.

(c) The location of shear surface(s) down each shaft would be noted and the extrapolation of such failure planes between adjacent shafts made.

(d) Shafts would greatly aid interpretation of seismic traverses already undertaken. Additional runs could then be attempted.

(e) Water inflows into shafts would be noted so that a crude measure of the slide mass permeability can be calculated. The permeability of the colluvium should be known if relief drainage is to be installed at the site.

(f) Piezometers installed following excavation of shafts would reveal whether ground water within the slide mass is perched. A number of piezometers should be installed in each shaft at varying levels within the slide, at least one of which should be located below the shear zone. It is recommended that simple standpipe piezometers (small diameter PVC tubing) be used, as these may then be monitored by means of a probe, run down the hole, to determine levels at which shearing displacements occur.

(g) Soil samples within the shear surface(s) should be removed for laboratory classification and strength testing.

The most likely locations for shafts would be in the vicinity of survey markers 4 and 7 described in Section 2.4 as a zone of deeper seated slope activity. Sites adjacent to the railway and county road would provide ready access to equipment.

As material in the slide is unlikely to stand vertically for any length of time exploratory shafts will have to be cased.

Following excavation of exploratory shafts it is recommended additional seismic refraction traverses be undertaken. Sufficient velocity contrast between refractor and overburden would appear to make a more detailed investigation by seismic refraction methods warranted. Such investigations may be performed at little cost over terrain in which other subsurface techniques would require expensive access routes to be constructed. Seismic refraction surveys correlated with exploratory shafts should provide sufficient depths of overburden to enable detailed cross and long-sections of the slip to be drafted.

Studies to determine the extent of water infiltration into the slide mass from the unnamed stream bisecting the site are recommended. The studies could possibly be carried out with the assistance of the Water and Soil Division, Ministry of Works and Development.

Finally, a slope stability analysis based on the shape of the failure surface and utilizing the soil shear strength mobilized at the shear surface should be performed. The effect of differing remedial treatment on the safety factor may be calculated.

2.10 CONCLUSIONS AND RECOMMENDATIONS

Natural geologic and geomorphic slope forming processes have caused the formation of a large landslide at the site of the Ethelton Slip.

Rapid downcutting through rocks of the Torlesse Supergroup during the last 10-12,000 years producing oversteepened slopes, coupled with extensive mechanical and chemical weathering of the predominantly volcanic bedrock, were the immediate causes of failure.

The construction of the road and rail over the toe, ground fracturing due to seismic activity allowing water infiltration, and fluctuating hydrostatic pore pressures, are believed to trigger intermittent, downward and outward movement of part of the landslide towards the Hurunui River at the toe of the slope.

The landslide is believed to be a complex combination of a slip in colluvium, a compound slide, and a slump earthflow. Rotational movement is inferred by the appearance of a sunken upper region and a heaved foot.

A mantle of weathered colluvium underlies the ground surface over most of the slide. Having undergone considerable disintegration through mass movements within the slip, the colluvium is likely to be everywhere highly variable in both physical properties and thickness.

Two relatively small and isolated zones of movement were noted during geological mapping. An area of surficial slumping in slopes immediately above and below the county road is located some 100-200m southeast of the unnamed stream. The locality is worriesome to the extent that debris flows and slides have been known to block the road during heavy rainfall.

Larger in area and potentially more troublesome, the second region of movement occurs in a zone between the Nos. 4 and 7 survey marker stations. At this locality, waterfilled tensional cracks and scarps, and slickensided shear zones, infer deeper seated, possibly rotational movement of a large soil block. Both transportation routes occur within the zone. Movement can be expected to continue at this area.

Precise surveying of surface markers confirms the findings of field observations that movement of a soil block in the vicinity of the No. 4 survey station is taking place. In the 9.7 month period from 11.4.76 to 3.2.77, station No. 4 moved a total of 13.1cm in a downslope direction towards the Hurunui River at the toe of the slide.

A correlation of rate of precipitation with rate of movement on station No. 4 has not yet been established.

Though none was recorded, movement on the No. 7 survey marker was expected due to the considerable surface activity in the form of tensional cracks and shear zones in the vicinity of the station. A longer period of time may be required to delineate movement in this region.

Long term monitoring of stations over several years will be necessary to determine accurately the amount of rainfall received at the site above which activity in the slip becomes greatly accelerated.

As appears likely that rates of precipitation will be shown to control activity in the slide, a relief drainage scheme will possibly be shown to be the most effective remedial treatment (see Section 5).

The N.Z. Railways are advised to continue consultations with the Department of Geology and the School of Engineering, University of Canterbury, and the Ministry of Works and Development and the N.Z. Geological Survey, so that further site investigations and remedial measures may be properly undertaken.

SECTION 3: THE HAWKSWOOD CUT - BATTER INSTABILITY ALONG AN EXCAVATED RAILWAY CUTTING

3.1 INTRODUCTION

In mid 1975 the writer approached N.Z. Railways with a view to undertaking an appraisal of the batter stability problems at the Hawkswood Cut. An investigation involving engineering geological mapping, and laboratory testing was undertaken during the latter part of 1975. The results of this work were presented to both N.Z. Railways, in the form of a report (Grocott, 1975), and the University of Canterbury, as part of the writer's first year M.Sc. study. A summary of the 1975 studies are presented in parts 3.3 and 3.5 of this section. A concurrent theodolite survey was also undertaken, by G. Hunter, assistant engineer, N.Z. Railways. His surveys provided cross-sections and long-sections of the Cut, from which excavation volumes have been calculated.

Field investigations and theoretical slope stability studies, completed subsequent to this earlier 1975 work, are presented in parts 3.4 and 3.6 of this section.

The Hawkswood Cut, situated seven kilometres north of the North Canterbury township of Parnassus, is the largest excavated railway cutting on the Main North Line. Approaching one kilometre in length, the cutting has an original base width of 7m and attains a compensated gradient of 1:70. Both east and west batters, rising to respective heights of 18.0m and 19.4m, have similar slope angles of 45^{o}; major slope instability in a central 200m long section has increased the batter here to a near-vertical angle. The Cut runs in a general north northwest direction (Fig. 13, Plate 11).

Since landslide corrective action was first considered in 1952, several remedial measures have been proposed. These proposals, including one to elevate the track up to 2m above present base level on a series of piles, and a second, projecting enclosure of the cutting within a tunnel of semi-circular iron sheeting, have all been rejected. Engineering geological site investigations have therefore been undertaken to enable the current proposal, allowing for a reduction in slope angles along the Cut, and relief drainage, to be properly executed. Earthworks are expected to be undertaken in the 1977-78 construction season, or shortly after.

Much of the cutting, principally along the central section and northern approach, has been constructed through relatively flat land of subdued relief. However, relief above the southern approach is more pronounced, with natural slopes steepening to 35^{o} away from the cutting crests. A hummocky topography paralleling the western crest, resulting from the dumping of spoil during construction, leads to severe ponding through much of the year. A 1-4m high ridge of spoil back from and running parallel to the eastern edge tends to facilitate runoff into the east side of the Cut.

Two flumes, one running parallel with, the other down the eastern slope, and a 1m deep cutoff trench above the western crest, provide the only attempt at surface drainage. The cutoff trench was installed without levelling

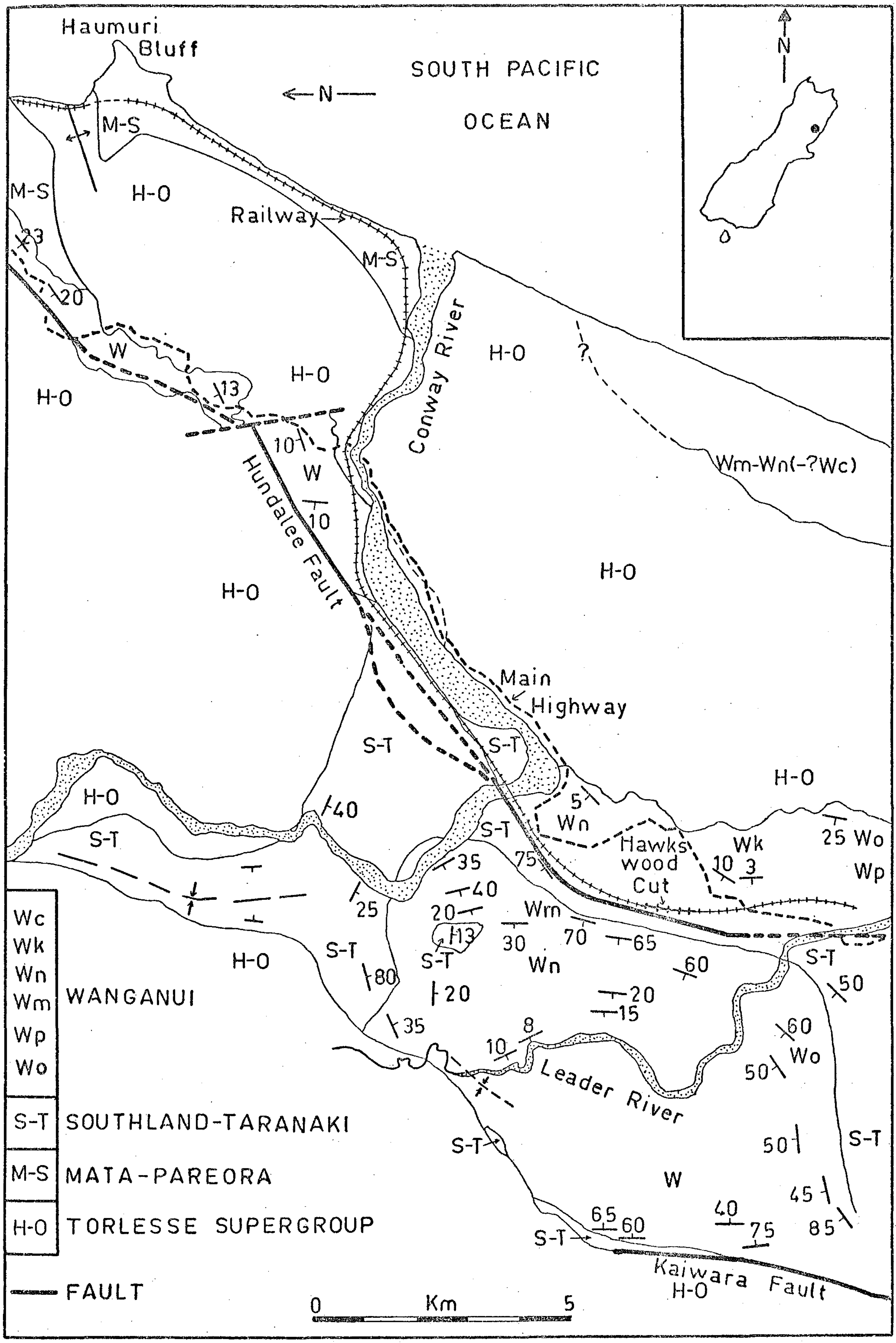


FIG. 13: LOCALITY AND GEOLOGICAL SKETCH MAP
(Geology after Warren, 1975)

however, and because of inadequate maintenance remains ponded along much of its course.

The following plan should be read in conjunction with section 3:

:Engineering Geological Longsection of the Hawkswood Cutting, North Canterbury.

3.2 REGIONAL GEOLOGY OF THE SITE

3.21 Stratigraphy

The Leader Basin, a depositional trough extending from the Hundalee Hills and the Hawkswood Range in the north and east respectively, to near the Lowry Peaks Range in the west and beyond the Waiau River in the south, comprises a mid Miocene - late Pleistocene (1-20 M. yrs) succession of predominantly marine sediments (Fig. 13). The basin, like many New Zealand Tertiary depositional environments, is flanked by indurated, highly fractured and complexly folded greywacke sandstone and argillite (mudstone) of the Torlesse Supergroup. Torlesse Supergroup sediments in this area are of probable Jurassic age (125-180 M. yrs) and form the basement rocks over much of central New Zealand.

The oldest rocks in the Leader sequence comprise non-indurated, blue-gray, marine silts of Southland to Taranaki age (mid-late Miocene). Overlying these silts is a highly variable succession of predominantly marine clays, silts, sands and conglomerates, of Wanganui age (early Pliocene - mid Pleistocene). These forms the rocks in the vicinity of the Hawkswood Cut.

PLATE 11. The Hawkswood Cut. A view looking south.

3.22 Structure

The Leader Basin is a north to northeast trending late Tertiary tectonic depression in which several distinct structural regions have been recognised.

A large structural feature, the Hundalee Fault, trending north to north eastwards through the Leader Basin, separates a gently northwest dipping Wanganui sequence in the vicinity of the Hawkswood and to the east of the fault, from steeply dipping Southland-Taranaki rocks to the west. Wanganui beds overlying these Southland-Taranaki sediments to the west of the Hundalee Fault dip steeply inwards from all directions. Further south of the Hawkswood, the rocks form a relatively simple synclinal structure.

In the west and northeast of the Leader Basin, the Kaiwara and Hundalee Faults, respectively, bring rocks of the Leader succession into faulted contact with Torlesse Supergroup sediments. Elsewhere, rocks of the two groups are in angular unconformity.

Uplift along the Hundalee Fault since the mid Pleistocene has been estimated at several hundred metres; recent movement along the fault is possibly illustrated by an inferred landslide immediately to the west of the Hawkswood Cut (Plate 13).

3.3 SITE GEOLOGY

3.31 Geological mapping. A succession of early-mid Pleistocene age sediments were identified along the Cut by geological mapping. The oldest beds in the sequence are non-indurated, slightly fossiliferous, blue-gray marine silts having an apparent dip of 14° to the northwest.

PLATE 13. An inferred landslide to the west of the Hawkswood Cut.

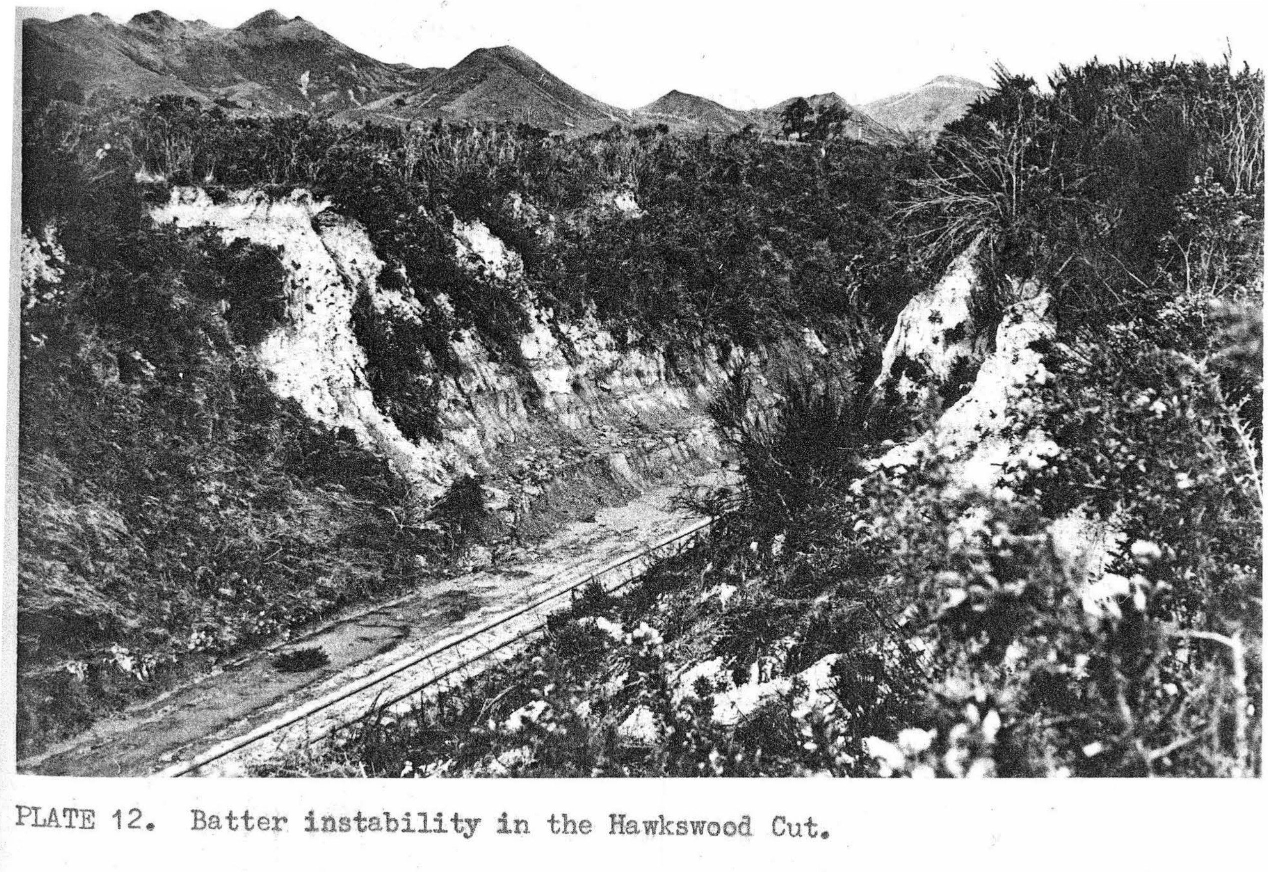


PLATE 12. Batter instability in the Hawkswood Cut.

These silts are exposed at the southern approach only.

Overlying the blue-gray marine sequence, and through which most of the Cut is constructed, is a conformable sedimentary succession displaying variations in depositional environment from marine through to estuarine. A typical sequence would include:

> Dark brown, laminated, carbonaceous clays; very thinly-medium interbedded, cyclic, blue-gray clayey silts and rusty brown sands; thickly bedded and occasionally cross-bedded creamy brown silty sands; rusty brown, blue-gray, thinly to thickly bedded, cemented, well-poorly graded gravels.

Warren (pers. com.) attributes many of these gravels to represent submarine debris-flow deposits, thus explaining their erratic lateral and vertical distribution throughout the cutting. Many gravels have scoured contacts with underlying sediments.

The attitude of this highly variable sequence is a gentle dip towards the northwest, though minor slumping after deposition has caused the dip to steepen in a few places. A 1m thick carbonaceous clay bed within the sequence dips at 3° in the direction of the northern approach. The succession has a minimum thickness of 18m.

Capping the esturaine-marine succession is a 3-4m thick sequence of gray alluvium and light brown loess-like silt.

3.32 <u>Description of soil types.</u> Engineering geological mapping has shown the cutting to be constructed in a wide range of "engineering" soil types. It was found convenient to classify these soils into five main units.

Each unit contains up to five beds consisting of soils with similar engineering properties. The description starts with the uppermost (youngest) soil unit.

Unit 1:

Bed A: Light gray, loosely compact, rounded GRAVELS* (2-60mm) and COBBLES (60-200mm) which are slightly layered and poorly graded. Contains some boulders (over 200mm) but few fines. Permeable. These gravels and cobbles are overlain by 1-2m or more of light brown, firm, silty LOESS.

Unit 2:

Bed B: Cream, soft, clayey SILT of moderate plasticity.

Bed C: Blue-gray, soft, clayey SILT of moderate plasticity. Slightly permeable.

Bed D: Rusty brown, friable, SAND (0.06-2mm). Discontinuous in extent.

Unit 3:

Bed E: Blue, gray and rusty brown, compact, moderately well graded, GRAVELS and COBBLES. Moderately permeable. At the northern end of the cutting this soil grades into SANDS and GRAVELS of Bed F and sandy silts of Bed G.

Bed F: Light brown, compact SAND. Not significant within this unit.

Bed G: Light brown, compact SAND and GRAVELS, grading laterally into Bed H.

Bed H: Light brown, compact, fissured, mottled sandy SILT, slakes on exposure after wetting.

* Definitions given are from the Geological Society Engineering Group Working Party Report (1972).

Unit 4:

Bed I: Dark brown, soft, fissured, fibrous organic CLAY of high plasticity. Impermeable.

Bed J: Blue-gray, very soft, fissured, clayey SILT of moderate plasticity. Impermeable.

Bed K: Rusty brown and blue-gray, soft, fissured, thinly layered horizons of SAND and clayey SILT. Slightly permeable. Towards the southern approach this soil grades into gravels and cobbles of Bed L.

Bed L: Two rusty brown, weakly cemented, permeable, GRAVEL and COBBLE horizons.

Bed M: Light yellowy gray, compact, poorly graded GRAVELS and COBBLES. Permeable. Lenses of weakly compact, light brown, SAND and SILT also.

Unit 5:

Bed N: Blue-gray, firm to soft, fissured, impermeable SILT which slakes on exposure. Moderately plastic.

3.33 Landslide Types

Three landslide types have been recognised:

(a) Slumps: N.Z. Railways engineers have informed the writer that the rotational movement of a slump-like feature was causing concern to the batter stability as far back as 1943. A slump, located 860m along the western slope, is inferred to represent this earlier instability. The feature extends laterally for 30m at base level, while disturbed ground within the slump extends to a 1m high scarp immediately below the batter crest. No sliding surfaces along which shear failure may have occurred were recognised.

Disturbed ground of a similar nature within the western slope, inferred to be a slump, occurs 920m along the Cut. A visit on 9 September, 1975 indicated ground surface fracturing with minor slumping in a 15m wide zone of slope debris in the eastern face, at the 1500m point. This area has continued to show minor movement throughout the 1976 winter.

No evidence of disruptions to rail traffic from rotational slumping of the cutting sides has been found. If left undisturbed, it is probable that slumps will remain in their equilibrium state and not cause concern to track safety in the future.

(b) Earthfalls: Many landslides are in the form of blocks of soil breaking away from the slope faces. These soil blocks are controlled by sets of steeply dipping fissures orientated parallel to the line. The blocks are irregularly shaped, ranging in size up to 4-6m diameter, occasionally much larger, and weigh up to several tonnes. When failure occurs, there is usually an immediate and rapid downward movement of the block away from the soil, with the resulting debris often coming to rest across the track (Plate 14).

Earthfalls are prevalent in a 200m long central section of the Cut, occurring in plastic, organic clays, blue clayey silts and rusty brown sands of Beds I, J and K, Unit 4. At this locality the soils outcrop above base level to a height of 10m. When failure occurs within them, undermining and progressive oversteepening of the overlying soils and slopes result.

To the north of this central area of instability, Beds I, J and K dip below track invert, while to the south they grade abruptly into more competent gravels and cobbles of Bed L, Unit 4.

(c) Earthflows: During high intensity or prolonged rainfall, oversaturated debris-gorged earthflows become mobilized to flow off the flanks of both batters (Plate 15). Such slips originate in the surficial 0.5-1m of slope material carrying debris rapidly onto the line, occasionally burying the track to a depth exceeding 1m.

Earthflows, by reason of their noncohesive slurry-like nature, are the most disruptive landslide type, often leaving surface clearance gangs no alternative but to allow locomotives to forge their own passage through the muck.

Small, slow-creeping earthflows also originate in a surficial 2-3cm of slope material during thawing after frosts.

3.34 Factors Inducing Batter Instability

(a) Rainfall: The association of high intensity or prolonged periods of rainfall with an increase in landsliding in general has been well demonstrated throughout the history of the Hawkswood.

During 1976, the worst general period of landsliding took place over 8 and 9 September, in which 35mm of rainfall fell producing a large number of small to large earthflows and earthfalls. However, delay to Main North Line rail traffic was only three hours.

Section 3.42 compares 1976 rainfall results with those of previous years.

(b) Runoff: Lack of surface drainage above, and a general slope direction towards the cutting, facilitates

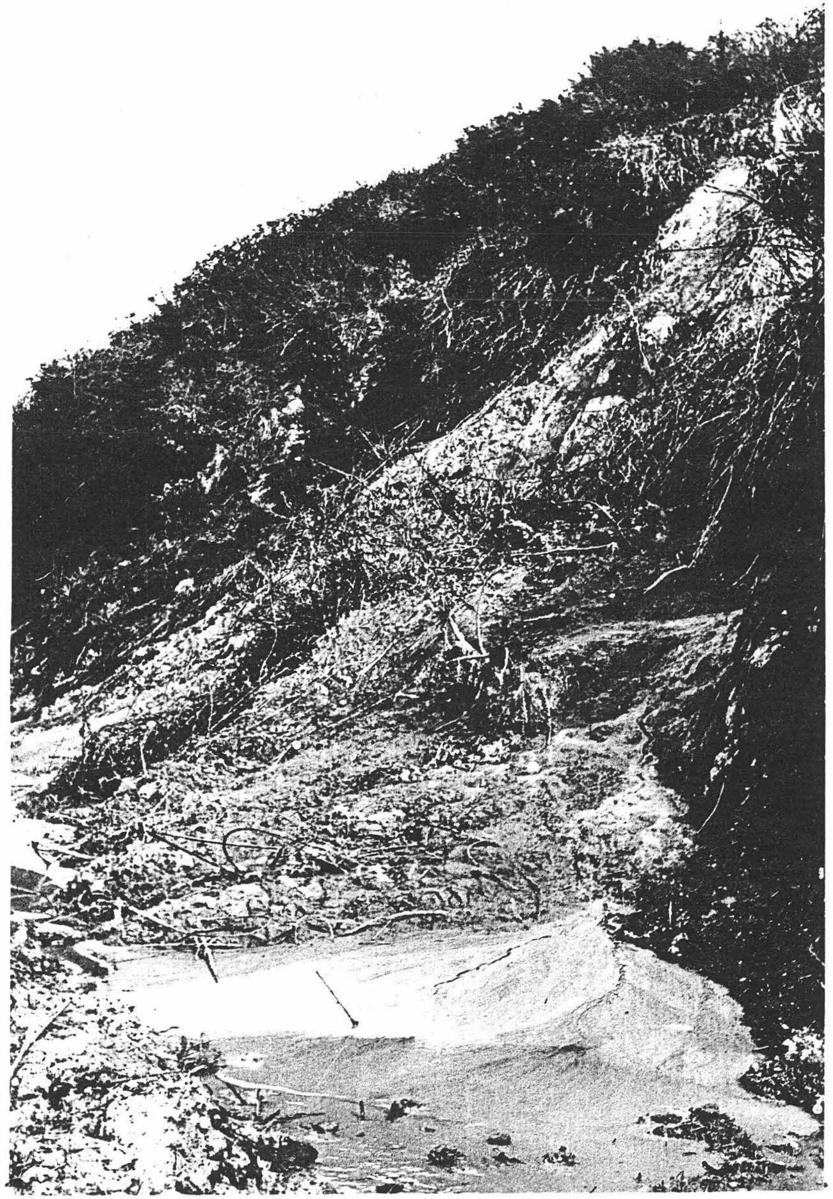


PLATE 15. Earthflow landslide.

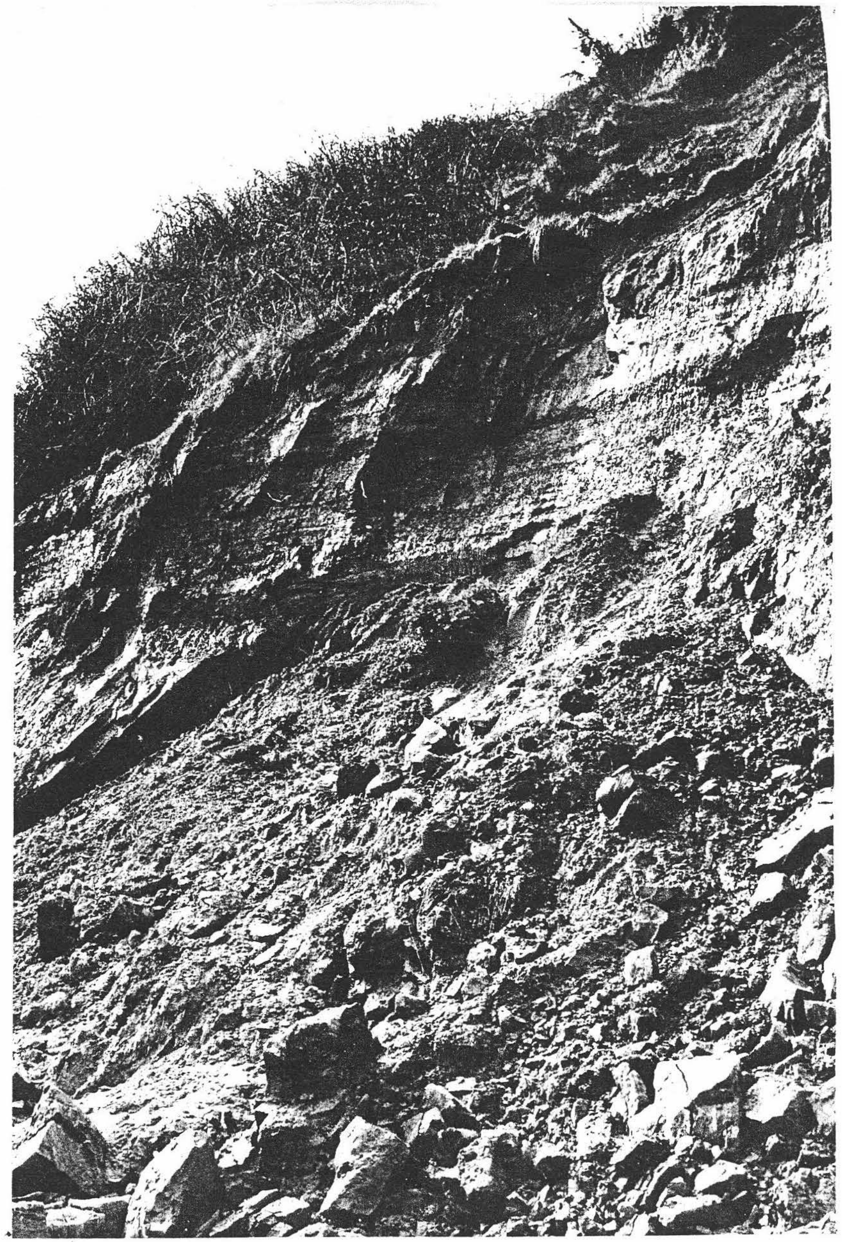


PLATE 14. Earthfall landslides.

runoff down slopes towards the track. Under these conditions, surficial slope material becomes rapidly saturated, pore water pressures rise, and the effective shear and normal stresses acting on potential failure planes increase and decrease respectively. When failure is imminent, earthflows become mobilized and flow downhill to cover the track.

(c) Relaxation fissures and seepage pore pressures. Running parallel to, and dipping steeply into and away from the track, are sets of fissures in many fine grained soils. These unfavourably orientated discontinuities, interpretated as relaxation joints formed through stress relief following construction, are a main cause of earthfall landslides.

For several weeks leading up to failure of a large soil block, a progressive development of its fissure from a well-defined tight crack to a 1-2cm wide gap running along the boundary of the block is noted. For small soil blocks, the mode of failure tends to be an instantaneous spalling of the block away from the slope face without a progressive development of its fissure.

Failures involving earthfalls occur predominantly following periods of intensified rainfall. Ponding on the ground surface above the cutting, and increased water seepage from slope faces, indicate a buildup of seepage pore pressures within finer grained soils. Excess pore pressures exerted on relaxation fissures are thought to be the principal triggering mechanism of earthfalls.

Long-term measurement of standpipe piezometers (Section 3.44) above the cutting will quantify this inflation of pore pressures following rainfall.

(d) Train-induced vibration frequencies: An earthfall involving more than 100 cu.m of spoil occurred on 2 August, 1976. At the time of failure heavy rainfall had not been recorded at the site for several weeks, and slope faces were noted as dry. Failure of the soil block was observed to take place several minutes after the passage of a Dj locomotive through the Hawkswood.

Train-induced vibrations may possibly be inferred as a triggering mechanism of earthfall landslides.

3.4 FIELD INVESTIGATIONS

3.41 Experimental Horizontal Drains

Three experimental horizontal drains were installed in December, 1976. The drains were installed by drilling a slighting-inclined borehole 4-5m above track level into the slope face for a distance of some 10-12m. After reaming each hole a 5.1cm I.D. (2 in) slotted P.V.C. tube was inserted into position.

0.2 litres/minute was recorded as the maximum water discharge from one drain after installation. Since then discharge has rapidly decreased, with only occasional trickles presently issuing from each drain. Silting of the tubing perforations may account for this drop, though it was felt during installation of the drains that the contractor lacked sufficient experience in this type of work.

3.42 Rainfall Measurements

A Marguis 600 Series rainfall gauge was installed in June 1976. The gauge has been assigned a N.Z.

Meteorological Service recording station number, H23633 (Hawkswood Bridge). Daily readings are taken by the district Railways Inspecting Ganger.

Meteorological Service recording stations H23632 (Hawkswood) and H23631 (Ferniehurst), located one kilometre northeast and five kilometres north of the site, have recorded rainfall since 1931 and 1941, respectively.

Table 4 summarises rainfall data from stations H23631, H23632 and H23633. During the period of this study, precipitation received within the catchment of the site was approximately equivalent to the district mean during the months of August, October, and November, 1976 and February, 1977. The months of September and December, 1976 and January, 1977 were wetter than the district mean, while July, 1976 was drier. No month recorded exceptionally heavy rainfall, though the worst period of landsliding at the site occurred during the wettest month, September.

3.43 Diamond Drilling

The Christchurch based Investigations Section, Ministry of Works and Development, were contracted to drill eight cored boreholes. The contract specified payment of a fixed sum, though if the sum was exceeded provisions for additional payments were allowed. Drilling was undertaken between July 28th and September 24th, 1976.

A truck-mounted Pioneer rotary drilling rig equipped with NQ size, split inner triple tube core barrels was used. Wire-line core recovery techniques were employed.

Drilling was undertaken for the following reasons:

(a) The continuation of certain gravel horizons along,

TABLE 4

MONTHLY RAINFALL FOR RECORDING STATIONS HAWKSWOOD (1931-76) AND FERNIEHURST (1941-76), AND WEEKLY RAINFALL FOR STATION HAWKSWOOD BRIDGE (JULY 1976-FEBRUARY 1977)

Figures in millimetres of rainfall

	HAWKSWOOD				FERNIEHURST				HAWKSWOOD BRIDGE				
Month	1	2	3	4	1	2	3	4	WK1	WK2	WK3	WK4	Total
January	80	281	7	55	92	296	13	62	57	17	56	0	130
February	92	483	17	202	67	177	24	134	4	24	35	0	63
March	98	443	10	45	92	268	10	55					
April	102	324	18	83	103	297	13	76					
May	138	490	20	75	121	416	29	69					
June	95	325	20	87	89	289	14	72				11	
July	122	568	8	104	120	546	16	119	40	11	18	1	70
August	114	503	4	171	124	484	6	231	18	54	1	49	122
September	81	429	12	177	65	400	12	149	51	77	6	16	150
October	89	255	3	127	91	270	11	108	6	50	36	0	92
November	80	320	0	96	80	298	6	107	18	12	0	56	86
December	84	268	9		84	233	20	211	25	22	77	18	142
Total	1175	1828	537		1128	1691	534	1393					

1 = mean (to 1975), 2 = high (to 1975), 3 = low (to 1975), 4 = 1976 figures.

and back from the present cutting, may allow a complex design involving cut and benching along the proposed construction.

(b) Subsurface drainage utilizing free-draining properties of gravel beds may be attempted if gravels persist laterally over a wide distribution.

(c) The installation of standpipe piezometers in boreholes will allow "before and after" construction monitoring of piezometric pressures at different locations and levels. As well, short term fluctuations following heavy rainfall and seasonal variations will also be monitored.

(d) Core samples to be used for soil testing.

Drill holes were sited at the following locations:

		Distance from Surveyed Centre Line	Distance from Southern Approach	R.L.* (metres) top	R.L.* (metres) bot.
DH1	east side	35m	750m	29.31	8.61
DH2	east side	65m	975m	36.48	9.08
DH3	east side	120m	1025m	39.02	15.62
DH4	east side	70m	1250m	39.01	17.8
DH5	west side	-45m	860m	34.51	12.31
DH6	west side	-65m	1050m	38.42	9.62
DH7	west side	-110m	1150m	39.38	17.98
DH8	west side	-65m	1300m	38.64	18.24

Core logging was undertaken at the Soils Testing Laboratory, M.W.D., Christchurch. Cores were removed from the split inner tube, wrapped in plastic and placed in wooden boxes for transport back to Christchurch. Logging was based

* All reduced levels are relative to culvert invert of R.L. 16.19m at 140.7074 Main North Line Kilometres.

on the report of the Working Party, Geological Society Engineering Group (1972), and McLean (1976).

Core recovery in fine grained soils was generally good, averaging 80-100%. Drilling through gravels usually produced only cuttings. The first 3m (10 feet) of each hole, drilled using continuous flight augers, also produced only cuttings.

Driller's records show small overnight water inflows into drill holes 3, 4 and 5. No records were kept for the other holes.

Results from the drilling programme include:

(a) Soils encountered in all drill holes have similar physical properties to soils mapped in the cutting, consisting generally of consolidated cohesive clays, silts and sands, and weakly cemented gravels and cobbles.

(b) Although soils encountered in drill holes have similar physical properties to soils mapped in the cutting, lensing out of gravel units separating the finer grained sediments prevented positive correlation of soils in the cutting with drill hole cores being made. For example, within the cutting, gravels of Unit 3, Bed G, commonly separated the soils, Unit 2, Bed C, and Unit 4, Bed J, two fine grained sediments of similar physical properties. Lensing out of the gravels away from the cutting resulted in their absence in cores of many of the drill holes. To distinguish between the two finer grained soils in drill cores was therefore found impractical; where a blue-gray, soft, clayey SILT occurs in cores, the soil has commonly been correlated with both Bed C and Bed J.

(c) Borehole to borehole correlation of soils logged in holes located on the eastern side was generally good. Correlation between drillholes on the western side was found more difficult.

(d) The highly discontinuous distribution of gravel and cobble horizons, both laterally and vertically, would make their use in any subsurface drainage impractical.

(e) The erratic distribution of gravels would also appear to rule out their use as near-vertical standing faces in a design based on cut and benching.

Logs of drills holes are given in Appendix 2. A tentative correlation between soils logged in each drill hole with soils mapped in the cutting is also given.

3.44 Installation and Measurement of Standpipe Piezometers

Simple standpipe piezometers were installed in all drill holes. In boreholes located on the western side (5, 6, 7, 8) a single, 5.1cm ID (2in) piezometer consisting of PVC tubing was inserted down each hole. Piezometers in boreholes 5, 7 and 8 consisted entirely of slotted PVC piping, while the piezometer in hole 6 utilized a small length of slotted tubing at the bottom of the piezometer coupled to unslotted PVC above. The piezometers were surrounded by a coarse sand/fine gravel ("pea-gravel") filter. As the PVC tubes were not sealed at preselected levels, they therefore record only an average water table level within each hole.

Installation of piezometers in drill holes located on the eastern side (1, 2, 3, 4) proved more difficult, as two or three piezometers were installed in each hole at various levels. Piezometers consisted of 1.3cm ID (0.5in) PVC tubing, the lower most 0.5m of pipe being slotted. A length of "terra firma" filter cloth wrapped round the slotted section of tubing acted as an inner filter. In each drill hole the piezometer preselected to record the lowest level was installed first, the slotted section of which was embedded in a medium sand filter. In drill hole 2, a grout composed of cement, bentonite powder and medium sand was used to seal the lower most filter from the next predetermined level. In holes 1, 3 and 4 bentonite pellets placed above the filter were used as a seal (Fig. 14).

Piezometers installed in drill holes 1, 2, 3 and 4 have been constructed to record piezometric pressures at preselected levels down each hole.

Water table levels were recorded using a simple battery-operated electrical probe coupled to a voltmeter on the end of a length of wire coil.

Piezometers were installed at the following depths below ground surface:

DH1	3.4-5.3m	clayey silt
	8.2-10.5m	clayey silt
	18.5-20.4m	clayey silt with sand
DH2	4.6-6.4m	clayey silt and sand
	25.2-27.4m	clayey sandy silt
DH3	9.5-11.3m	clayey silt with sand
	22.2-24.4m	clayey silt
DH4	4.6-6.6m	clayey silt with gravels
	10.6-12.6m	clayey sandy silt
	18.9-21.3m	sandy clay and silt with gravels
DH5	slotted 0-22.2m	
DH6	unslotted 0-27.4m	
	slotted 27.4-28.6m	
DH7	slotted 0-21.4m	
DH8	slotted 0-20.4m	

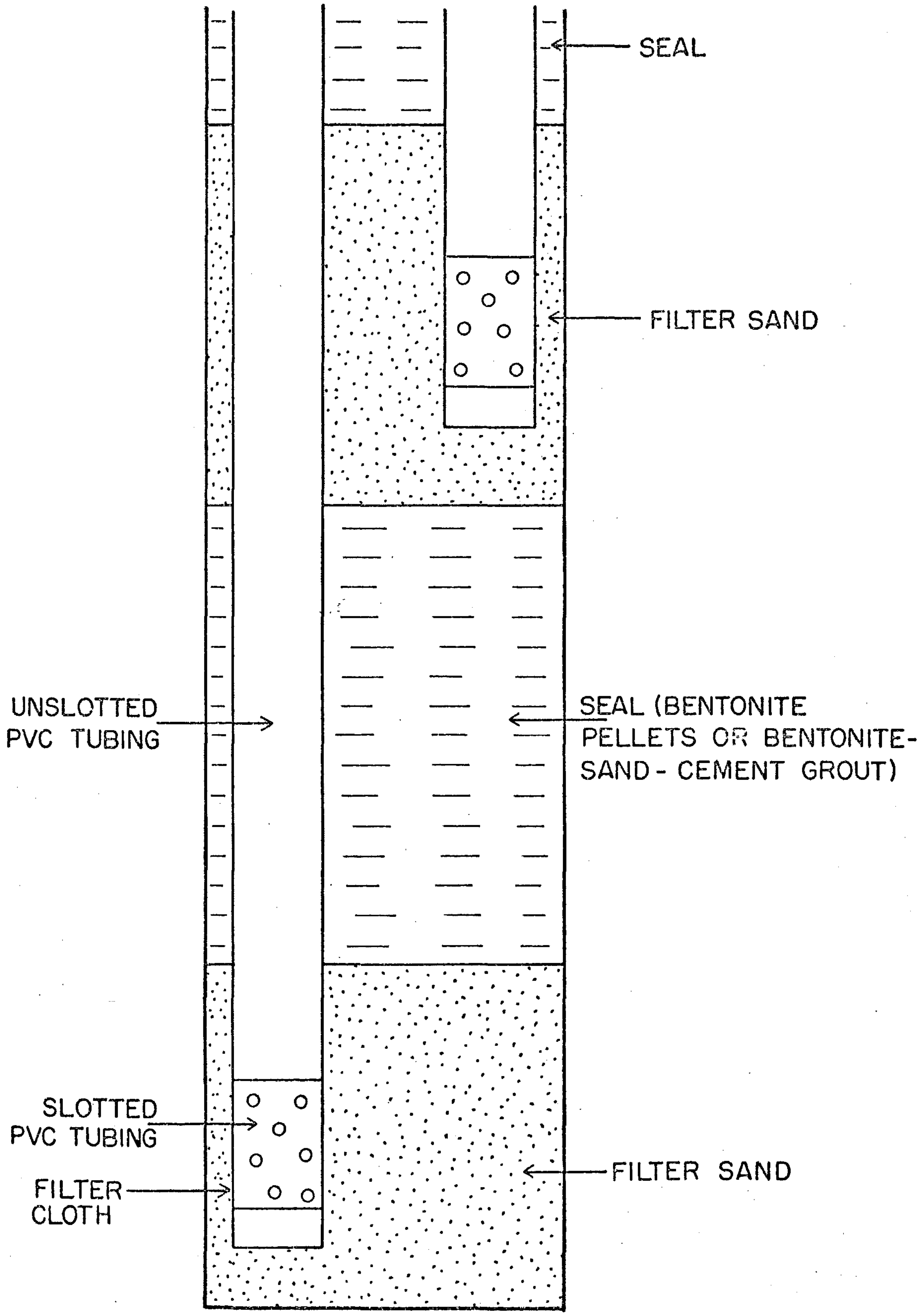


FIG. 14: STANDPIPE PIEZOMETER INSTALLATIONS

Results of piezometer measurements are given in Table 5.

TABLE 5

RESULTS OF PIEZOMETER MEASUREMENTS

	Level	28.12.76	4.2.77
DH1	shallow	4.64m	4.59m
	intermediate	9.54m	9.47m
	deep	blocked	blocked
DH2	shallow	1.29m	1.81m
	deep	5.59m	5.79m
DH3	shallow	1.82m	2.18m
	deep	8.54m	8.68m
DH4	shallow	5.77m	5.77m
	intermediate	10.49m	10.86m
	deep	15.40m	15.62m
DH5		18.55m	19.00m
DH6		7.49m	8.05m
DH7		10.97m	11.23m
DH8		10.51m	12.57m

No conclusions can yet be drawn concerning a correlation of fluctuating pore water pressures with precipitation recorded at the site. A longer period of monitoring will be needed to effect a correlation, if any exists.

However, results of piezometer measurements taken from the east side do infer a complex pore water pressure regime in which several phreatic surfaces exist at various depths below the ground surface. As the soils in the vicinity of the cutting are extremely heterogeneous in composition, such a complex regime could possibly be expected.

The highest recorded water table on the western side, in drill hole 6, is directly attributable to ponding in the vicinity of the hole.

3.5 LABORATORY TESTING

3.51 Methods and Results

Disturbed and undisturbed fine grained soil samples from exposures in slope faces and drill hole cores were tested.

(a) Classification tests. Classification tests used in the study include:

- moisture content;
- plastic limit, liquid limit, plasticity index;
- dry density, natural density, saturated density;
- grain size;
- uniaxial swelling;
- slake durability.

Mass and dry density were determined by weighing a known volume of soil. Saturated density was calculated from the following equation:

$$p_s = 1 + p_d \quad (1 + \frac{1}{SG})$$

where:

p_s = saturated density

p_d = dry density

SG = specific gravity

Uniaxial, or one-directional, swelling was measured by recording strain along a single axis of an unconfined circular cylindrical soil specimen after its complete immersion in water.

Slake durability is an index test designed to measure the relative accelerated weathering resistance of a soil or rock to weakening and disintegration resulting from a standard cycle of wetting and drying (Franklin and Chandra, 1972). The procedure involves rotating oven dried samples for 10 minutes in a circular test drum of standard mesh size, with the drum half immersed in a water bath at 20°C. The erodable products of slaking and disintegration pass through the mesh into the water bath. Two cycles of wetting and drying are usually performed. The slake durability index, Id, is given by:

$$Id = \frac{\text{final dry weight}}{\text{initial dry weight}} \times 100\%$$

Other classification tests were performed according to New Zealand Standard Specifications (1976). Grain size analyses were determined by the Soils Laboratory, M.W.D., Christchurch.

(b) Permeability. The falling head permeameter method, a measurement of the rate of movement of a column of water suspended above a circular cylindrical soil sample through the soil, was used to determine thecoefficient of permeability, k, of several fine grained soils.

The coefficient of permeability is given by:

$$k = \frac{al}{At} \log_{10} \frac{H1}{H2} \times 2.3 \text{ cm/sec.}$$

where:

a = area of water column above sample in square millimetres

l = thickness of sample in millimetres

A = area of sample in square millimetres

H1 = initial height of water column above base of sample in millimetres

H2 = final height of water column above base of sample in millimetres

t = time interval in seconds.

Typical permeability values given by Terzaghi and Peck (1967) are:

degree of permeability		k (cm/sec)
high	gravels	10^3-10
medium	clean sands	$10-10^{-2}$
low-very low	fine sands, silts, clay	$10^{-2}-10^{-6}$
impermeable	homogeneous clay	$10^{-6}-10^{-8}$

(c) X-ray diffraction analyses. Scanning was carried out over a field between 3^{o} and 36^{o}. Tables 6 & 7 present results of tests (a) to (c).

(d) Shear strength testing. The force required to mobilize sliding of saturated soil on a potential failure surface is given by its shear stress, S. The force acting across a failure surface, the total normal force 6, tends to resist sliding (Fig. 15). The two stresses are related by the following equation, usually termed the Mohr-Coulomb stress equation:

$$S = C_u + 6 \tan \phi_u$$

where:

S = shear stress

6 = total normal stress

C_u, ϕ_u = cohesion and angle of internal friction, both in terms of total stress.

If the normal stress resisting sliding is reduced to zero, an initial driving stress is needed to initiate sliding. This initial stress is the cohesion. It can be seen that as the normal stress increases, the shear stress

TABLE 6

SOIL PROPERTIES, HAWKSWOOD CUT

Test	Bed C	Bed G	Bed I	Bed J	Bed K	Bed M	Bed N
Natural moisture (%)	15.0	17.3	32.2	22.6	19.9	14.9	21.3
Dry density (T/m^3)	1.78	1.74	1.39	1.67	1.69	1.76	1.80
Mass density (T/m^3)	2.09	2.04	1.74	2.05	2.00	2.02	2.11
Saturated density (T/m^3)	2.11	2.08	1.87	2.05	2.05	2.10	2.12
Uniaxial swelling (strain %)	10.3	8.3	11.6	7.8	1.2	1.2	3.3
Liquid limit (%)	34.6	28.1	94.6	29.1	25.7	27.4	32.8
Plastic limit (%)	20.2	18.3	62.8	22.3	25.1	20.5	19.7
Plasticity index (%)	14.4	9.8	31.8	6.8	0.6	6.9	13.1
Permeability (cm/sec)	$3x10^{-6}$	$1x10^{-3}$		$2x10^{-6}$ $5x10^{-6}$		$3x10^{-4}$ $4x10^{-6}$	
Slake durability 1st cycle	12.3	0	63.0	0	0	17.1	91.3
Slake durability 2nd cycle	0		52.5			8.6	49.0
X-ray diffraction	mt+v, mc, c, f	(v), q, f	mt, k, mc, q, f	k, (v), q, f			

mt: montmorillonite mc: mica

v: vermiculite c: calcite

k: kaolin f: feldspar

q: quartz

TABLE 7

SOIL PROPERTIES HAWKSWOOD CUT, DETERMINED BY

CHRISTCHURCH SOILS LABORATORY, M.W.D.

Bore Hole	Depth	Grain Size clay	silt	sand
1	10m	38	56	6
3	10m	31	64	5
	17m	31	64	5
	24m	34	62	4
5	6m	23	67	10
	17m	25	67	8
7	3m	21	51	28
		LL=26,	PL=17,	PI=9

needed to promote failure also increases.

If a water pressure U acts on a failure surface, the total resisting force 6 is reduced by an amount (6 - U) to the effective normal stress 6'. The shear stress causing failure is now expressed as:

$$S = C' + 6' \tan Ø'$$

where:

$6' = 6 - U$

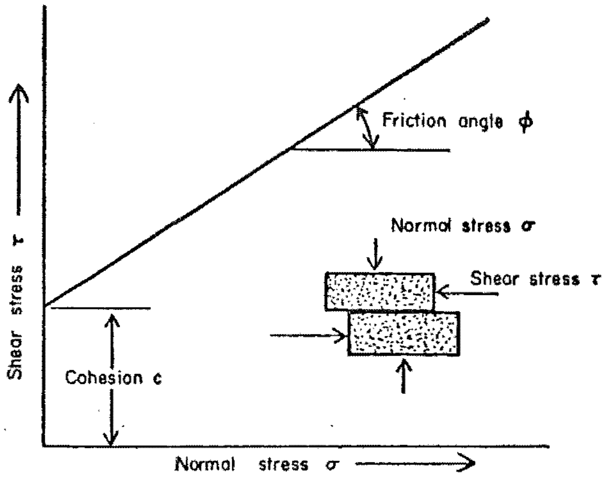
C' and Ø' = cohesion and angle of internal friction, both in terms of effective stress.

When shear displacement occurs along a potential failure surface of a soil under a constant total or effective normal stress (6 or 6'), the shear stress causing sliding will rise rapidly until the shear strength of the failure plane is exceeded. This maximum shear stress is called the peak shear strength. With increasing shear displacement along the sliding plane, the cohesion of the soil decreases and the shear stress drops to a level where it remains constant. This reduced shear stress is defined as the residual or ultimate shear strength of the slide plane (Fig. 15).

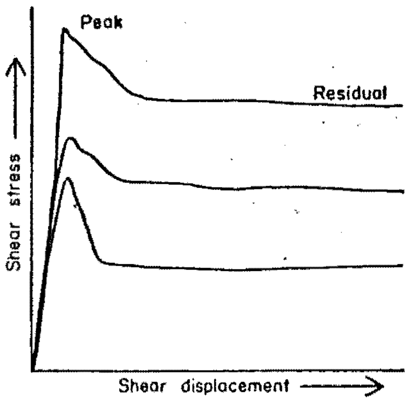
A soil tested under several normal stresses will produce a range of peak and residual shear strengths. This range may be graphed on a plot of shear versus normal stress (Fig. 15). A line drawn through each range will have a slope ϕ, termed the angle of internal friction of the soil. A given soil will therefore have both a peak ϕ_f and a residual ϕ_r angle of internal friction.

A series of drained triaxial shear tests was undertaken to obtain strength parameters for use in slope stability analyses.

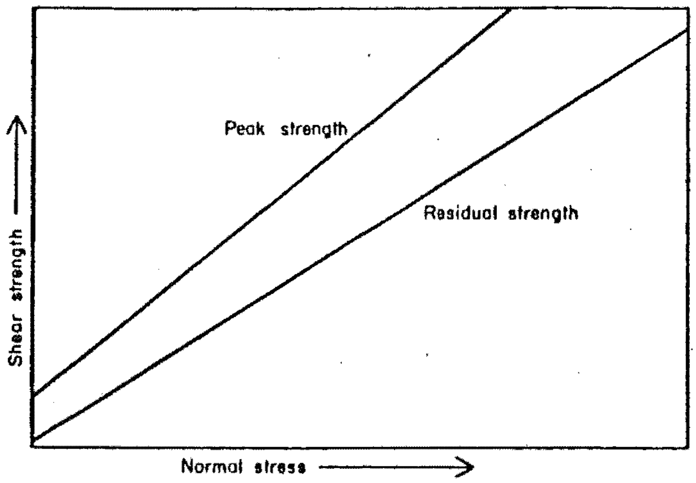
(i) Four drained triaxial shear tests, without pore pressure measurements, were undertaken and the resulting peak effective stress parameters (C' and ϕ') calculated. Circular cylindrical soil samples of size 3.6cm diameter x 7.2cm were triaxially consolidated under normal stresses of 300, 450 and 560 k Pa, these stresses being in the range of the existing overburden pressures, until a plot of log-time versus volume-change remained effectively constant. An axial loading rate of 6.1×10^{-4} cm/min. was then applied, this being sufficiently slow to allow dissipation of pore pressures under the given soil permeability conditions. Filter paper strips placed down each specimen facilitated drainage. Two to three days was the average time to peak failure during testing. At any time during loading, the



RELATIONSHIP BETWEEN THE SHEAR STRESS τ REQUIRED TO CAUSE SLIDING ALONG A DISCONTINUITY TO THE NORMAL STRESS σ ACTING ACROSS IT.



SHEAR STRESS VERSUS SHEAR DISPLACEMENT



PEAK AND RESIDUAL STRENGTHS MEASURED FROM STRESS-DISPLACEMENT

FIG. 15 : AFTER HOEK AND BRAY, 1974

deviator stress $(\sigma_1'-\sigma_3')$ is shown to equal:

$$(\sigma_1'-\sigma_3') = \frac{\text{stress (dial reading)x proving ring factor}}{\text{specimen area (A)}}$$

where:

σ_1' = major stress

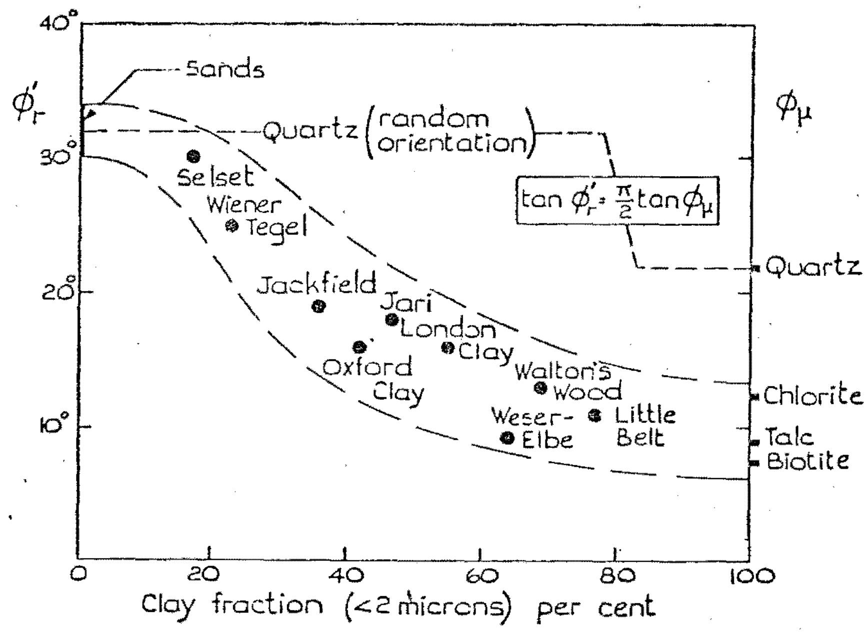
σ_3' = minor (cell pressure) stress

$A = \dfrac{\text{Vo - volume change}}{\text{Lo - length change}}$

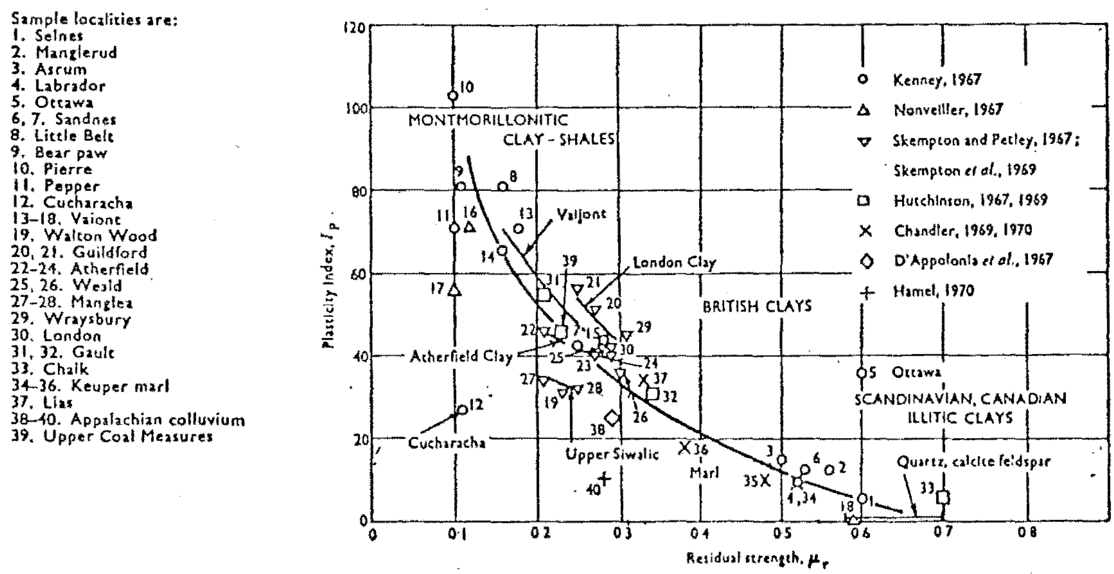
such that, Vo and Lo represent the original specimen volume and length, respectively.

The deviator stress of each specimen tested will peak at failure, the peak value depending on the confining pressure used. Results of each test are presented in terms of Mohrs Failure Envelopes in which a plot of shear stress versus effective normal stress is graphed (Appendix 3). Table 8 summarises the results of shear strength testing.

(ii) Skempton (1964) presented a broad correlation of the residual shear strength of a given soil with the percentage of clay-sized particles in the same soil. Voight (1973) presented a similar correlation of residual shear strength with the Atterberg plasticity index of a given soil (Fig. 16). Table 9 presents residual shear strengths of fine grained Hawkswood Cut soils based on clay percentage and plastic limit.



after Skempton (1964)



after Voight (1973)

Fig. 16. Relationship of residual shear strength with clay percentage and plasticity index for a given soil.

TABLE 8

PEAK EFFECTIVE SHEAR STRENGTHS, HAWKSWOOD CUT SOILS

Unit	Bed	Cell Pressure (k Pa)	Deviator Stress ($\sigma_1'-\sigma_3'$)	Delta vol. Consolida-tion (%)	Strain (%)	Strength
2	C	300	735	2.67	22.7	C'=10k Pa
		450	955	2.65	21.8	Ø'= 33°
		560	1320	4.00	11.7	
3	G	300	960	2.05	13.3	C'=0 k Pa
		450	1240	2.38	20.5	Ø'= 36°
		560	1530	5.39	11.2	
4	I	300	1100	4.50	16.5	C'=40k Pa
		450	1625	4.73	24.8	Ø'= 39°
		560	1640	5.80	27.7	
4	J	300	870	2.92	10.6	C'=10k Pa
		450	1340	1.65	11.4	Ø'= 36°
		560	1615	5.99	10.1	

TABLE 9

RESIDUAL SHEAR STRENGTHS, HAWKSWOOD CUT SOILS, BASED ON CLAY PERCENTAGE AND PLASTICITY INDEX

Material Unit	Bed	Voight	Skempton	Average
2	B		24^{o}, 27^{o}	26^{o}
2	C	26^{o}	27^{o}	27^{o}
3	G	29^{o}		29^{o}
4	I	18^{o}		18^{o}
4	J	31^{o}	24^{o},25^{o},27^{o}	27^{o}
4	K	33^{o}		33^{o}
4	M	31^{o}		31^{o}
5	N	27^{o}		27^{o}

3.52 Discussion and Conclusions to Laboratory Testing

(a) Insitu moisture contents of fine grained soils are at, or slightly below, corresponding plastic limits under average winter conditions. Field examination has shown that during heavy rainfall many of these soils exceed their plastic limits with a resulting drop in field strength from firm-stiff to very soft-firm.

(b) Plasticity indices are generally low to moderate. However, field "determinations" suggest higher plasticity indices than laboratory values, possibly due to soil moisture contents approaching their plastic limits under field conditions.

(c) Density and slake durability values are similar to other New Zealand non-indurated, consolidated, fine grained marine sediments. On the slake durability classification system of Franklin and Chandra, most soils would be classified as low-very low; that is they slake to a large extent. Mass density values are only fractionally below each soil's saturated density under average winter conditions.

(d) Uniaxial swelling coefficients are low-moderate, though trace concentrations of the swelling clay mineral montmorillonite are reflected in those soils having the highest swelling coefficients. Many soils tested for swelling and slake durability were found to disintegrate rapidly on immersion in water, suggesting slaking rather than swelling to be the more important property.

(e) Fine grained soils tested for grain size are clayey silts with small proportions of sand.

(f) Permeability values range from low - effectively impermeable, generally reflecting soil grain size. Soils with low permeability values also exhibit high earthfall-type instability, the inference being their inability to dissipate excess seepage pore water pressures.

(g) Effective residual strength parameters as determined by the methods of Voight and Skempton are, with one exception, only marginally lower than respective peak parameters. Residual shear strengths are consistent with similar normally consolidated clayey sandy silts of low-moderate plasticity.

(h) The four soils tested under drained effective stress conditions were all normally consolidated. The effect of shear, as in an earthquake, on normally or lightly overconsolidated soils is to cause the material to contract and reduce its voids (Hawley et al, 1975). If the material is saturated, this reduction in voids will produce excess positive pore water pressures, a condition responsible for the failure of many natural slopes, cuts and embankments. Development of positive pore pressures is greatest in silts and fine sands having low permeability; clays resist shear by virtue of their cohesion, while coarse sands and gravels allow dissipation of pore pressures. Heavily overconsolidated soils in shear undergo an increase in voids and the development of negative pore pressures, a condition inducive to slope stability.

Changes in stress conditions in cohesive soil are also experienced following excavation of slopes in cuttings. The short and long term stress conditions of such slopes have been discussed by many workers (Schuster, 1968-Skempton, 1970 - Vaughan and Walbanke, 1973 - Chandler, 1974 - Chandler and Skempton, 1974 - Eigenbrod, 1975).

A slope unloaded during excavation undergoes a process of stress relief resulting in an immediate decrease in pore pressures in the vicinity of the construction. Slope failures in such cuttings may occur both in the short term (end-of-construction-case), where shear stresses caused through excavation may induce failure, or in the long term, steady seepage case. Short term slope failures are referred to as the $S = C_u$, $\phi_u = 0$ condition. In the long term, slope failures are attributed to a progressive lowering of the effective normal stress in the soil. This is due to a gradual build up of pore water pressures from their initial, low, end-of-construction value to a condition of steady seepage in which pore pressures are in equilibrium with stresses acting in the slope. In addition, the effects of weathering and discontinuities may hasten this progressive weakening.

Slopes cut in heavily overconsolidated material may induce initial large negative pore pressures in the soil following excavation. Dissipation of these negative pressures may take many years, thus explaining the phenomena of delayed failure of slopes cut in London and Upper Lias clays.

Slopes cut in normally or lightly overconsolidated cohesive soils (Hawkswood Cut) however, reach their steady seepage state and a condition inducive to failure in a much shorter period of time. Thus, for excavations through normally or lightly overconsolidated soils, drainage is essential for the long term stability of the slope.

The short term slope failure where rupture of the soil is due to excavation-induced shear stresses is analogous to the undrained total stress laboratory test. The drained effective stress test in which no build up of seepage pore pressures occur is analogous to the long term slope condition. The inability of soils to dissipate excess seepage pore pressures during high intensity rainfall is the usual triggering mechanism of long term slope failures.

Most natural slopes are in the long term steady seepage state.

3.6 SLOPE STABILITY ANALYSES

3.61 Introduction

Slope stability analyses are undertaken in practical engineering studies to "serve as a basis either for the redesign of slopes after failure or for choosing slope angles in accordance with specified safety requirements in advance of construction" (Terzaghi and Peck, 1967). All engineering structures are designed to include at least some margin of safety throughout their performance. In soil mechanics, this margin of safety is termed the safety factor, and is used to guard against the ultimate failure of soils beneath both natural and cut slopes.

The safety factor against ultimate failure of a slope is determined by a slope stability analysis in which the ratio of the forces resisting failure (shearing resistance) to those forces causing failure (shearing stress) are measured. When a value of unity is reached failure is imminent. The resistance of a soil to rupture is usually expressed in terms of the Mohr-Coulomb shear strength equation (section 3.51): $S = C + 6 \tan \emptyset$.

Advances in stability analyses have reached the stage where planar and rotational slides in soils, and planar and wedge failures in rocks, may now be analysed (Bishop, 1955 - Morgernstern and Price, 1965 - Janbu, 1973 - Hoek and Bray, 1974). Falls, many types of flows and certain complex landslides involving multiple failure mechanisms have not yet received the study enabling empirical analyses to be computed.

In this slope stability study the limiting equilibrium method of analysis is used. All methods of limit equilibrium analysis are based on the premise that "the forces tending to induce sliding are exactly balanced by those resisting sliding" (Hoek and Bray, 1974). A condition of incipient failure along a continuous slip surface of known or assumed shape is postulated. The safety factor of the slope is based on the shear strength properties of the material forming the sliding surface.

A second method of stability analysis, a more theoretical approach, is that based on the techniques of stress analysis and finite element analysis. These techniques are founded on the continuum approach to rock and soil mechanics in which the effects of stress on the rock mass as a whole are considered (Mahtab et al, 1970). However, recent advances in finite element analysis now allow the effects of rock discontinuities on slope stability to be considered.

The choice of correct shear strength parameters in limit equilibrium analyses is frequently considered. If pre-existing slip surfaces exist, in which reactivation of the landslide by earthworks is to be designed against, the residual effective stress (C_r', ϕ_r') must be used (Skempton, 1970).

For design against first-time slope failures in the short term, the $S = C_u$, $\phi_u = 0$ condition should be used.

Back analyses of a large number of first-time failures in the long term have shown that actual field shear strength parameters have values ranging near the laboratory-obtained peak effective stress (C', ϕ') to approximately the residual effective stress (C_r', ϕ_r').

Actual strength values calculated by back analysis of long term first-time failures in normally consolidated and non-fissured overconsolidated soils are only slightly less than the theoretical peak. First time slides in fissured over-consolidated clays correspond to strengths approaching the residual (Skempton, 1970).

Slope stability analyses were undertaken at the Hawkswood Cut to identify those ground conditions most affecting overall slope stability. Ground conditions affecting stability considered in the analyses include changes in, pore water pressure, slope and sliding angle, and soil shear strength parameters.

Two methods of limit equilibrium stability analysis were employed:

(a) Slip surfaces pertaining to arcs of circles were analysed according to the Bishop Simplified Method (Bishop, 1954) and the Fellenius (Conventional, U.S.B.R., Ordinary) Method of slices (Krey, 1926).

(b) A method based on Hoek and Bray (1974) analysing planar slides in rock slopes was adapted for use.

3.62 Slip Circle Analysis

Slip circle stability analyses were performed on the Ministry of Works and Development, Vogel Computing Centre (V.C.C.) IBM 370/168 computer using the ICES-LEASE I programme. The analyses were run from the Christchurch terminal.

LEASE I (Limiting Equilibrium Analysis in Soil Engineering), a subsystem of ICES (Integrated Civil Engineering System), is a computer programme designed to perform stability analyses according to the Bishop Simplified and Fellenius Methods of slices (Fig. 17). The programme is designed to handle any given soil and ground surface profile and pore water pressure condition.

LEASE I has several basic limitations:

(a) Only slip surfaces that are arcs of circles can be analysed.

(b) The simplified methods used do not satisfy equilibrium

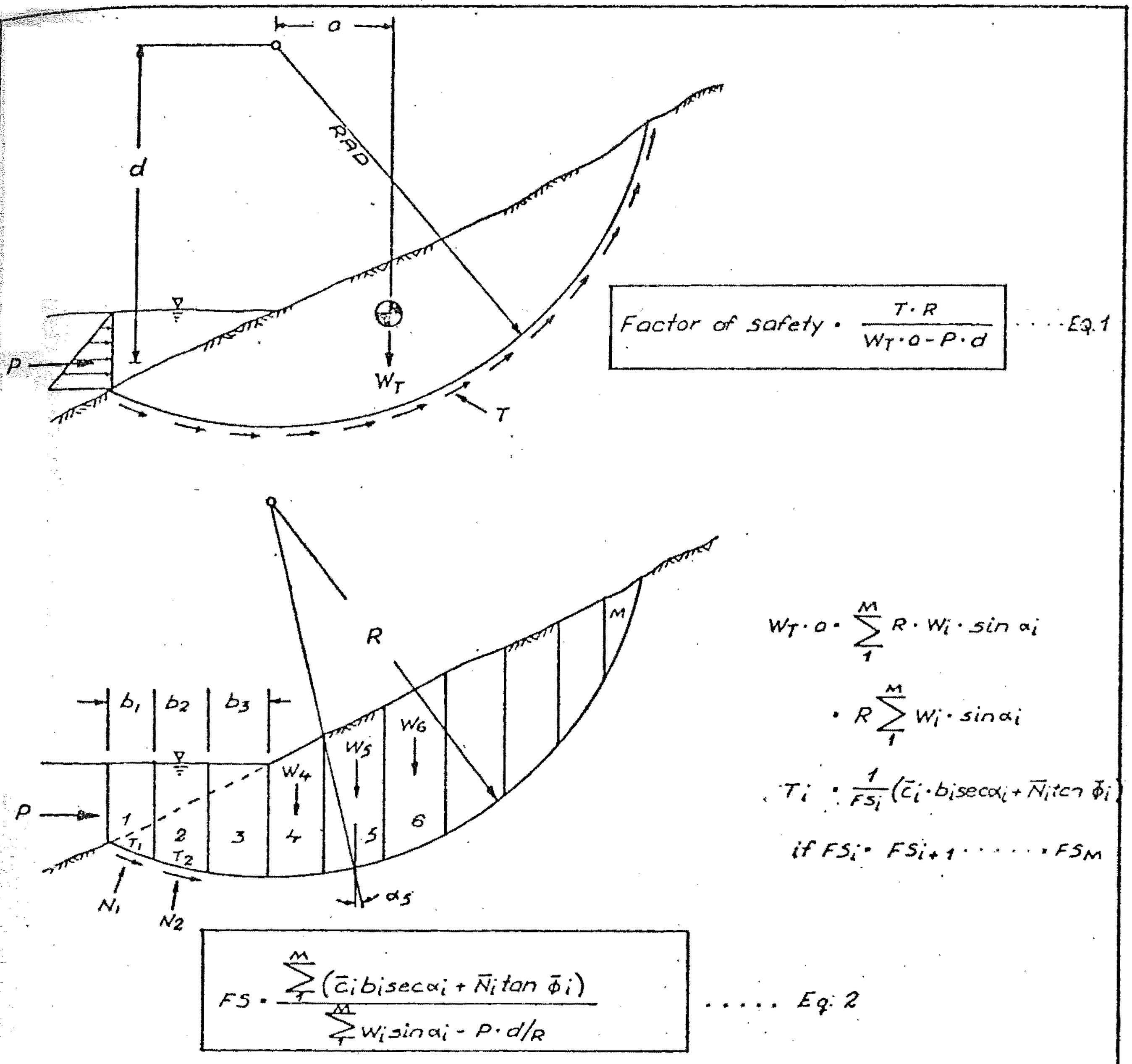


Fig. 17. *THE SLICES METHOD* (from Bailey and Christian, 1961).

conditions.

(c) As in all other methods of limit equilibrium analysis only two dimensional problems can be solved.

The programme will locate the minimum safety factor in the following ways:

(a) Each of a specified set of trial centres on a grid system will be analysed for the circle with radius having minimum safety factor.

(b) A search routine may be initiated to locate the minimum radius and its centre.

(c) In addition, explicitly defined circles may be analysed.

The procedures used in analysis of the Hawkswood include the following:

(a) A cross-section, with a slope angle of 45° from the horizontal, through a point 1200m along the western slope was used for the analysis (Line ABD, Fig. 18).

(b) A single, simple phreatic surface based on an average water table was introduced.

(c) Initially a coarse grid with a specified set of trial centres was instigated to locate a relative minima. From this first analysis three types of circular failures were observed:

(i): Small slip circles involving less than 0.5m of slope material beneath the slope face.

(ii): Intermediate size failures involving approximately half the slope face.

(iii): Large failures of the whole slope in which slip circles passed through or near the batter toe.

(d) A control was initiated so that only slip circles passing close to or through the batter toe were analysed. The phreatic surface was reduced in level (line AQD on Fig. 18) until a safety factor approaching 1.00 was attained.

(e) The effects of a 6m deep backfilled trench (line AOQD) and a 6m long horizontal drain (line APQD) on the phreatic surface were considered and the safety factors calculated.

(f) The slope angle was reduced to 30° at the same cross-section (line ACD, Fig. 18). Analyses of the same phreatic surfaces, adjusted for slope reduction, were performed (lines ARD, AORD, APRD).

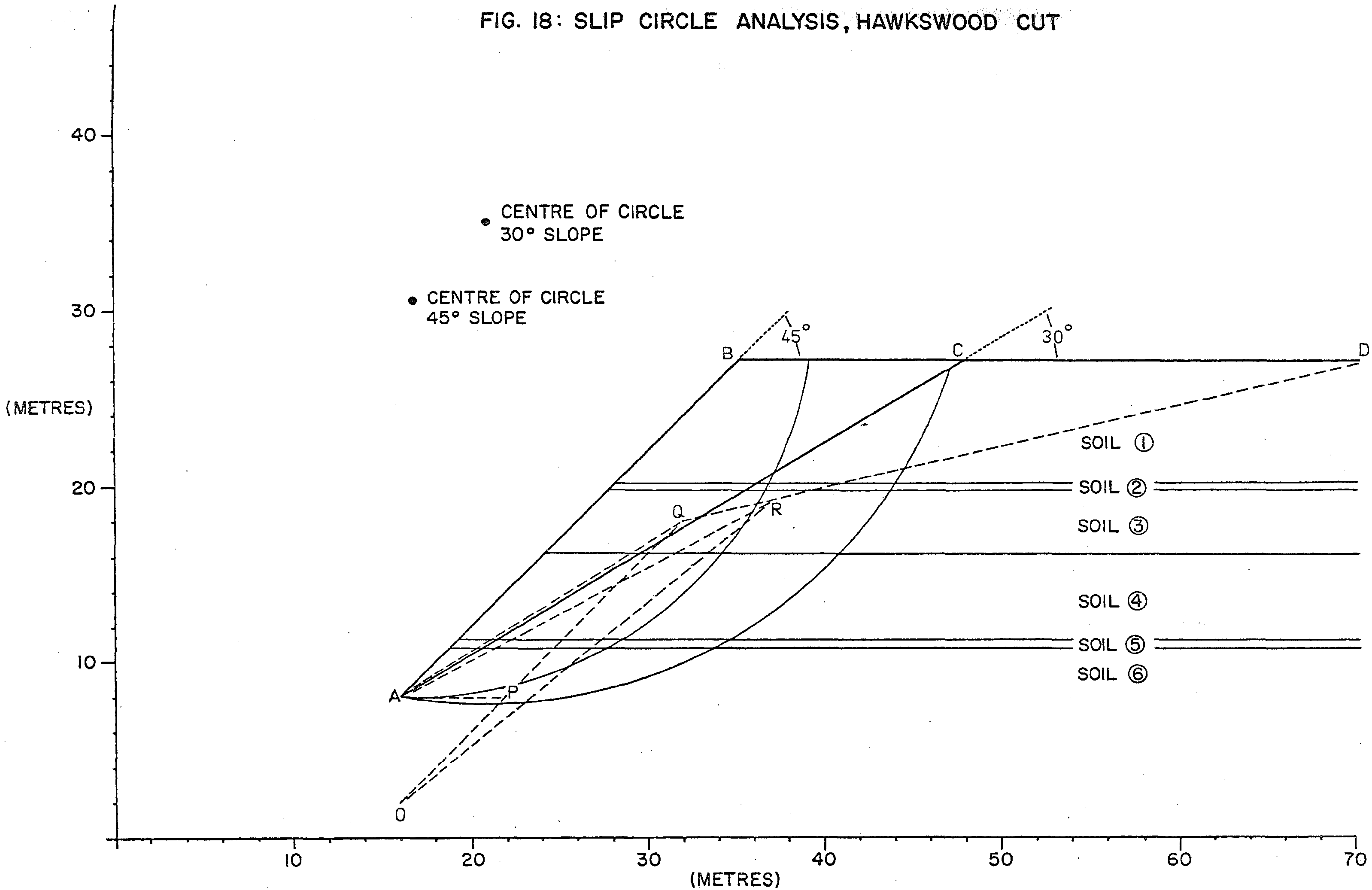
(g) Peak effective shear stress parameters were used in the calculations. Insufficient time prevented analyses using residual values and additional cross-sections based on drill hole data. Soil strength parameters used were:

Soil	Density (T/m^3)	C' (T/m^2)	Ø' (Deg.)
1	2.54	1.0	45
2	2.11	1.02	33
3	2.11	1.02	33
4	2.54	1.0	45
5	1.87	4.08	39
6	2.04	1.02	36

Gravels, being weakly cement-bound were given a small but finite cohesion.

As the analyses were performed to identify parameters most affecting overall stability, <u>relative</u> differences, not

FIG. 18: SLIP CIRCLE ANALYSIS, HAWKSWOOD CUT



absolute values of safety factor, should be read from the results in Table 10.

TABLE 10

FACTORS OF SAFETY AGAINST SLIP CIRCLE FAILURE

	F.S 45° slope	% increase with drainage	F.S 30° slope	% increase with drainage	% increase with slope reduction
No drainage	1.08		1.21		12
Backfilled trench	1.23	14	1.44	19	17
Horizontal drain	1.23	14	1.45	20	18

3.63 Discontinuity Controlled Planar Slides

Hoek and Bray (1974) describe an analysis of planar slides in rock slopes (Fig. 19). Failure, controlled by a single discontinuity intersecting the rock mass, is generally considered a specific case of the more usual wedge-type failure.

While the plane failure analysis does not usually describe the actual field conditions existing in a slope, it does have advantages in that the sensitivity or vulnerability of a particular slope to changes in pore pressures, shear strength, and slope angle and height can be demonstrated.

The conditions for plane failure to occur are:

(a) Failure is by sliding only.

(b) $\emptyset F$ greater $\emptyset P$ greater $\emptyset$.

(c) Release surfaces must define the lateral boundaries of the slide though these will provide no resistance to sliding.

(d) A tension crack, with or without water, must be considered in either the ground above the slope crest or in the slope face.

Assumptions which make the plane failure analysis possible are:

(a) Both sliding surface and tension crack strike parallel to the slope face.

(b) Water to a depth Z_w fills a vertical tension crack Z.

(c) The sliding surface has a shear strength defined by the Mohr-Coulomb equation:

$$S = C + 6 \tan \phi$$

(d) Water enters the sliding surface at the base of the tension crack and seeps along the slide plane to drain freely at the toe of the slope.

The factor of safety against plane failure is given by:

$$F = \frac{CA + (W \cos\phi P - U - V \sin\phi P) \tan \phi}{W \sin\phi P + \cos\phi P}$$

where:

C and ϕ = effective shear strengths

A = base area of the sliding block

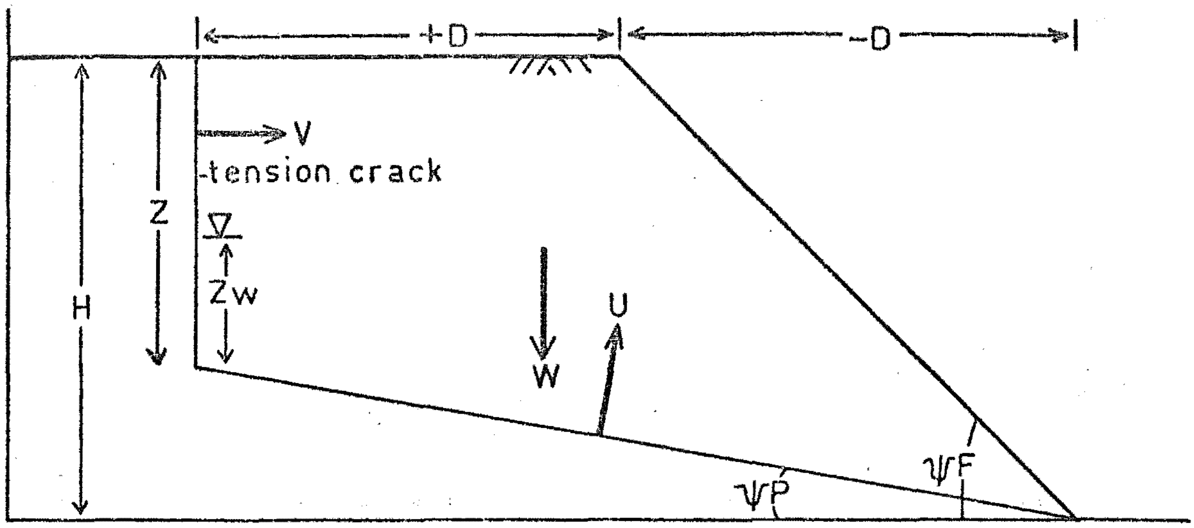
W = weight of sliding block

U = uplift force on slide surface due to water pressure

V = force due to water pressure in the tension crack

ϕP = angle of sliding.

Hoek and Bray's method of plane failure analysis has



W = weight of the sliding block
U = uplift pressure due to water pressure on sliding surface
V = force due to water pressure in tension cracks
ψF = slope angle
ψP = slide angle
H = slope height
Z = tension crack height
Zw = water height in tension crack
D = distance of tension crack from slope crest

FIG. 19: PLANE FAILURE ANALYSIS, HAWKSWOOD CUT (AFTER HOEK AND BRAY, 1974)

been adapted by the writer for use in analysing the sensitivity of earthfall slides (section 3.33b) to changed ground conditions. The validity for use of such an analysis in obtaining absolute values of safety factor for slope failures in heterogeneous soils may be questioned. However, bearing in mind soils are fissured, and that sliding of soil blocks along fissures probably does occur, the analysis was performed merely to reinforce as critical those factors affecting slope sensitivity obtained by the slip circle analysis.

In addition, the plane failure analysis includes the effects of a tension crack on stability, a factor which clearly should be considered in most stability analyses.

A simple Fortran computer programme was written for use on the University of Canterbury's Burroughs B6718 computer to analyse the sensitivity of earthfalls to changed ground conditions. For a slope of given height, the programme has the ability to:

(a) Change the effective normal stresses on the slide plane from peak to residual.

(b) Increment in value the angles of sliding and slope.

(c) Progressively change in location a tension crack from a position above the slope crest into the slope face.

(d) Increment the height of water in the tension crack from dry to completely full.

The residual and peak effective stress parameters used represent an average value for all soils. The parameters used were:

	Peak		Residual
cohesion (k Pa)	10		0
angle of internal friction	35		28.5
sediment density (T/m^3)		2.1	
water density (T/m^3)		1.0	

Figures 20a, 20b, 20c, 21, and 22 illustrate the results of many hundreds of computer-calculated plane failure stability analyses.

3.64 Assessment of Slope Stability Results

Conclusions from the slip circle analysis are:

(a) Reduction of slope angles from 45° to 30° without additional relief drainage will result in a small though measureable increase in the margin of safety.

(b) A reduction of hydrostatic pressures in a 45° slope will give a higher (albeit only slightly) safety factor than a 30° slope without drainage.

(c) Satisfactory drainage installed in a 30° slope will result in a larger percentage increase in safety factor than for a similar hydrostatic reduction in a 45° slope.

(d) A 34% increase in safety factor could theoretically be attained by the reduction of a 45° slope without drainage to one of 30° with adequately functioning drainage. Such an increase in safety margin is considerable.

(e) Only small reductions in hydrostatic head are needed to effect the increase in safety factor presented in Table 10.

Conclusions to be drawn from the plane failure analyses are:

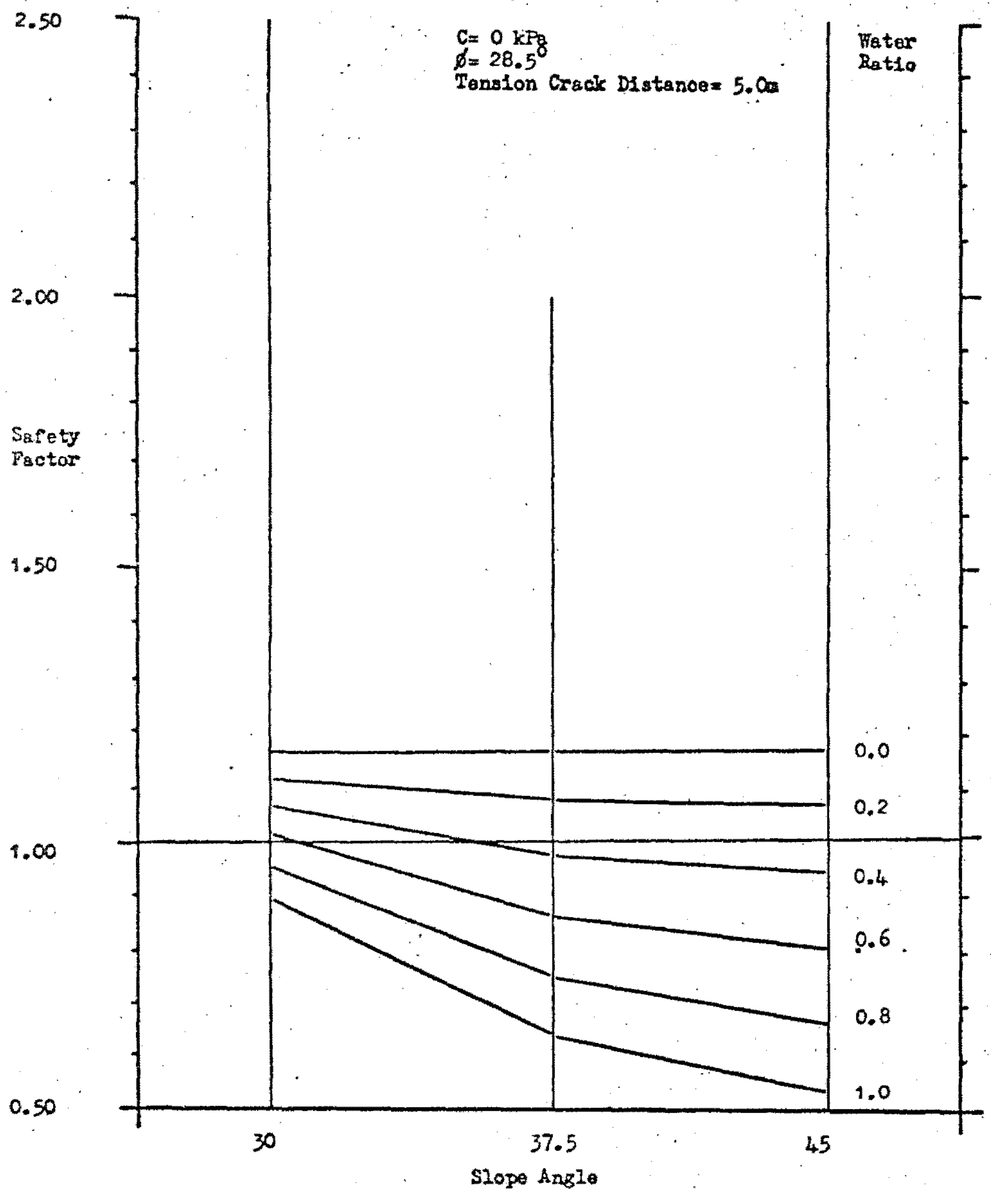
(a) The effect of an increasing head of water in a tension crack at the head of a slide is to lower the factor of safety (Fig. 20). Drainage must therefore be considered in any remedial action.

(b) Using peak shear strength values in the plane failure analysis, the safety factor of the slope will increase with a reduction of the batter angle (for the same water ratio). This result generally holds true when residual shear parameters are used in the analysis, except when the tension crack contains water and is located below the slope crest; here a slight decrease in safety factor is effected with a reduction in batter angle (Fig. 20).

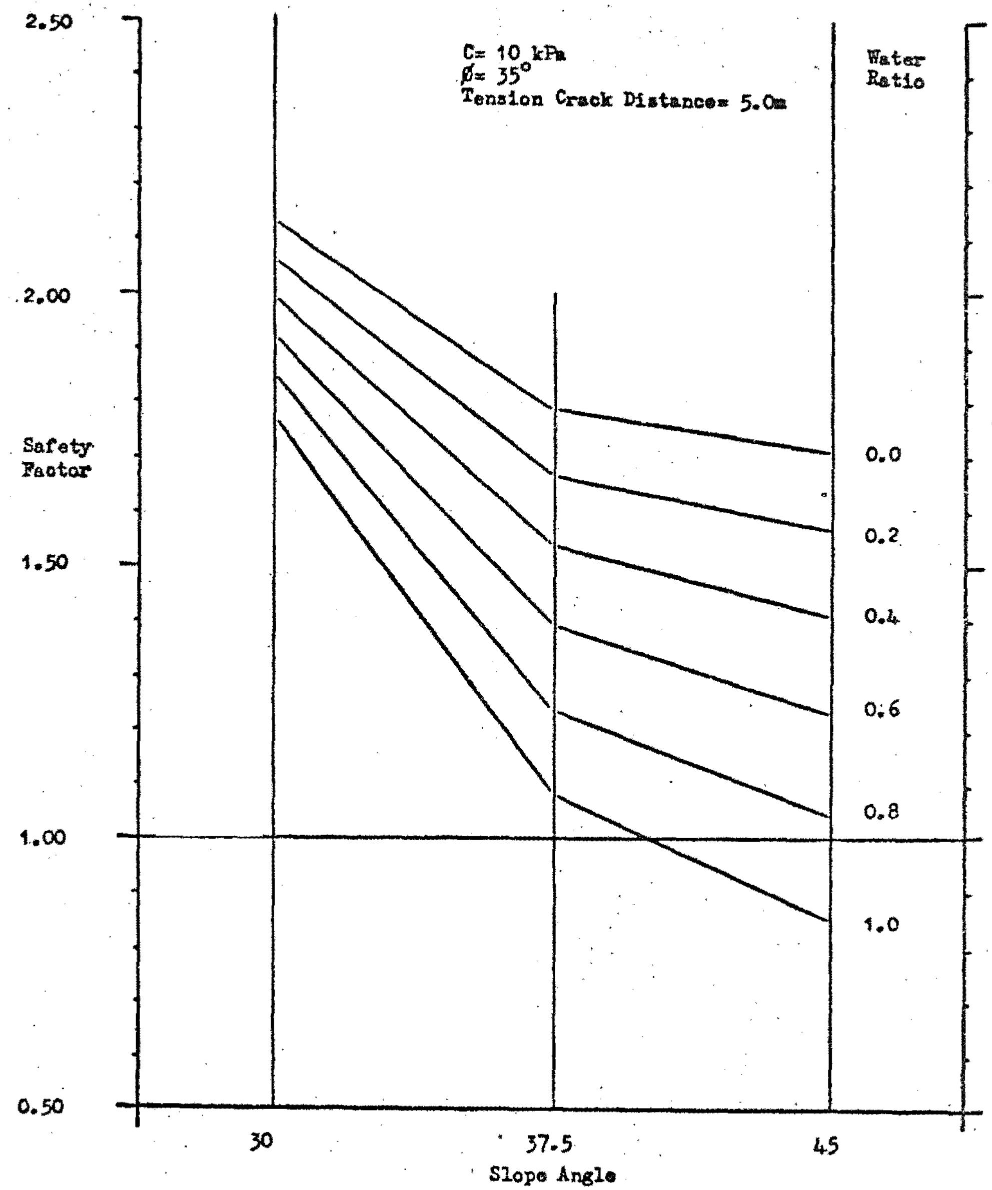
(c) The results of (b), above, indicate that the reduction in the weight forces (W) causing sliding (as a result of decreasing the batter angle) will generally exceed the corresponding reduction in the normal stresses (σ') resisting sliding (as a result of lowering the overburden pressure); however, should the tension crack contain water and be located below the slope crest (and the residual shear strength be mobilized at the slide surface), the reduction in the resisting stresses (σ') as a result of lowering the batter angle will exceed the corresponding reduction in the driving forces (W). Thus, the importance of keeping the slope dry through drainage is again illustrated.

(d) For a given slope angle, factors of safety decrease with increasing angle of sliding (Fig. 21).

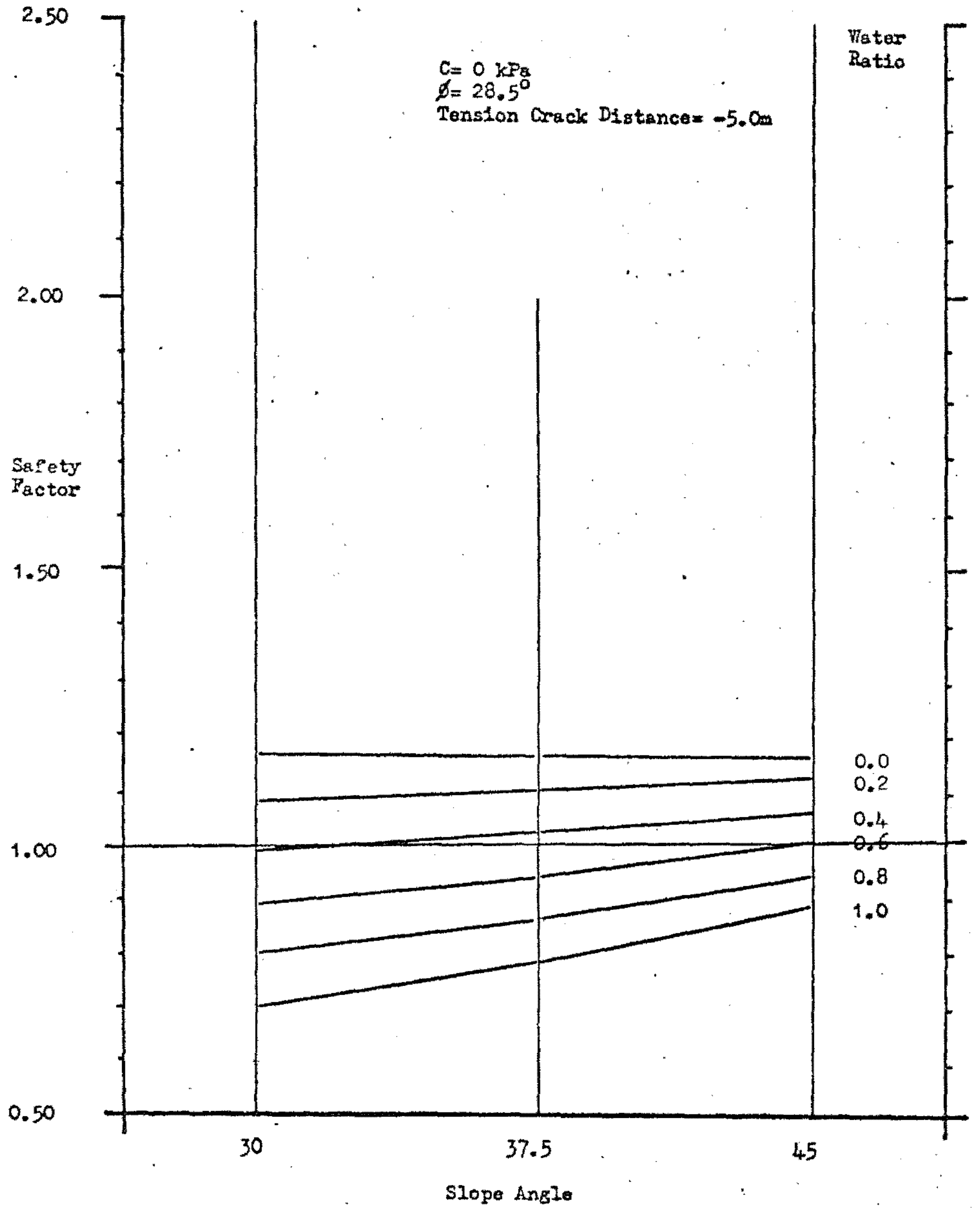
(e) For a given slope angle, a tensional crack located at the slope crest will produce a minimum safety factor; safety factor rises with increasing distance down the slope face and back from the slope crest (Fig. 22).



C= 0 kPa
Ø= 28.5°
Tension Crack Distance= 5.0m
Water Ratio
Safety Factor
2.50
2.00
1.50
1.00
0.50
0.0
0.2
0.4
0.6
0.8
1.0
30
37.5
45
Slope Angle



C= 10 kPa
Ø= 35°
Tension Crack Distance= 5.0m
Water Ratio
Safety Factor
2.50
2.00
1.50
1.00
0.50
0.0
0.2
0.4
0.6
0.8
1.0
30
37.5
45
Slope Angle


2.50
2.00
1.50
1.00
0.50
Safety Factor
C= 0 kPa
Ø= 28.5°
Tension Crack Distance= -5.0m
Water Ratio
0.0
0.2
0.4
0.6
0.8
1.0
30
37.5
45
Slope Angle

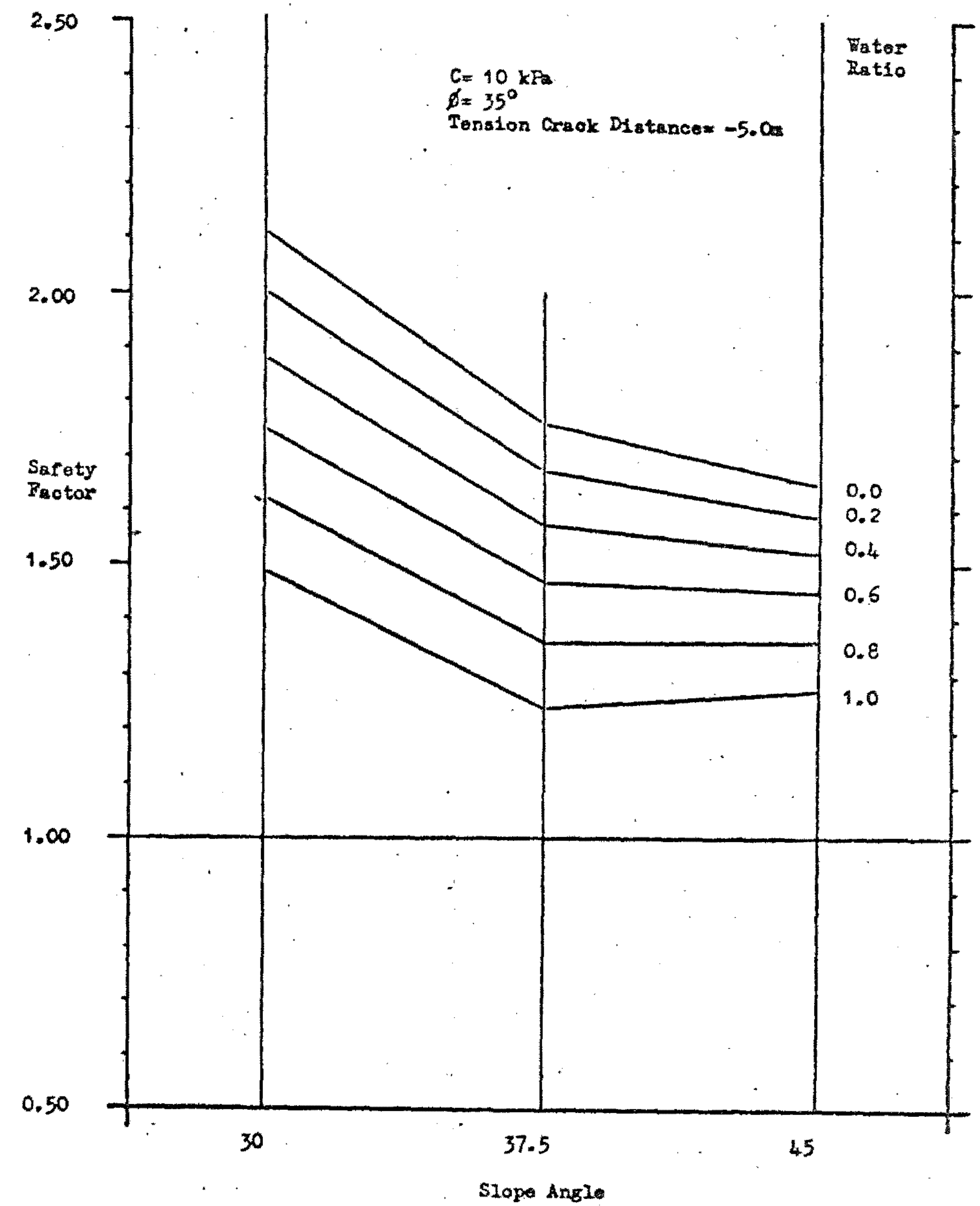
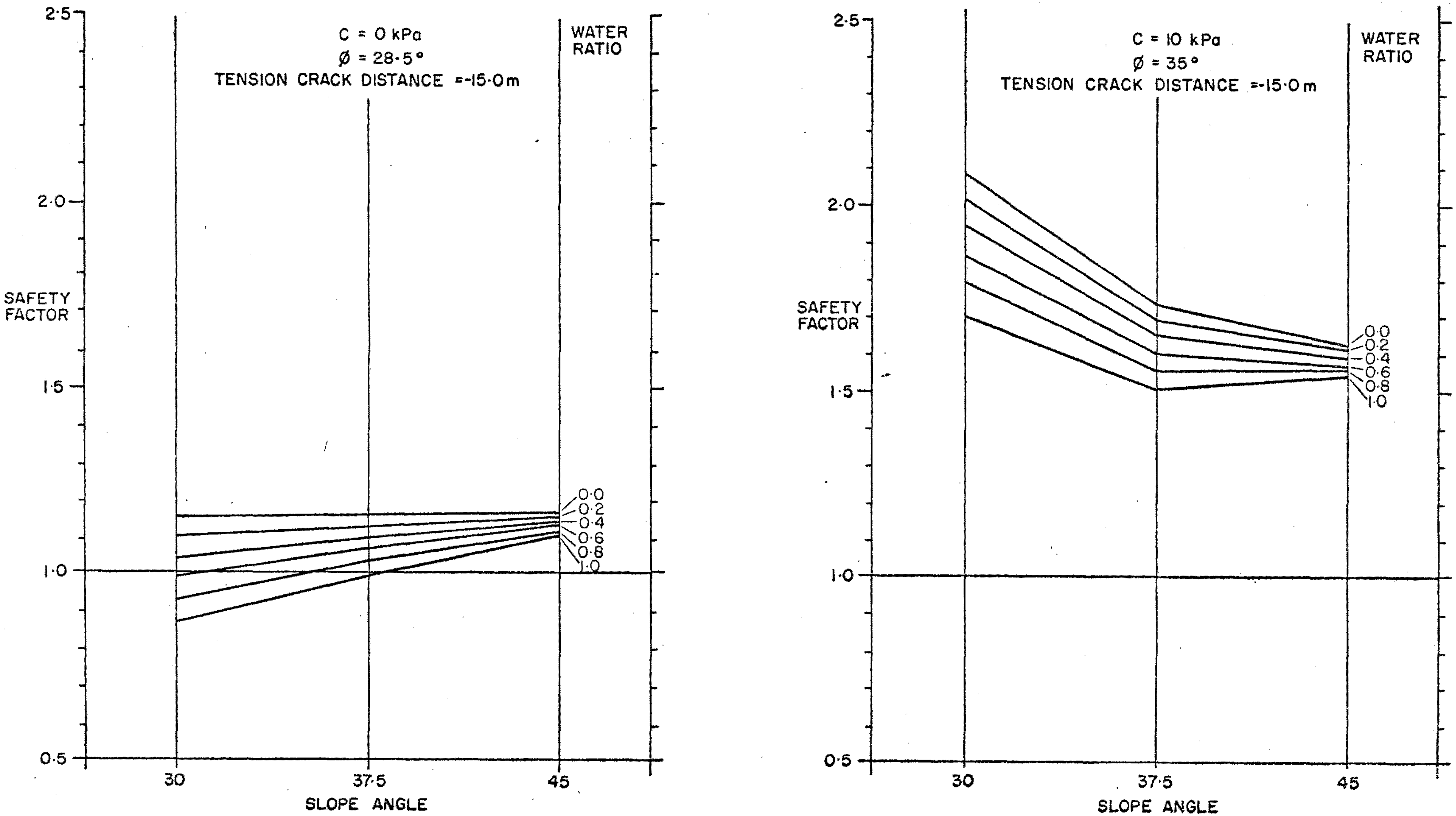

2.50
2.00
1.50
1.00
0.50
Safety Factor
C= 10 kPa
Ø= 35°
Tension Crack Distance= -5.0m
Water Ratio
0.0
0.2
0.4
0.6
0.8
1.0
30
37.5
45
Slope Angle

FIG. 20c



2·5
2·0
1·5
1·0
0·5
C = 0 kPa
Ø = 28·5°
TENSION CRACK DISTANCE =-15·0 m
WATER RATIO
SAFETY FACTOR
0·0
0·2
0·4
0·6
0·8
1·0
30
37·5
45
SLOPE ANGLE
2·5
2·0
1·5
1·0
0·5
C = 10 kPa
Ø = 35°
TENSION CRACK DISTANCE =-15·0 m
WATER RATIO
SAFETY FACTOR
0·0
0·2
0·4
0·6
0·8
1·0
30
37·5
45
SLOPE ANGLE

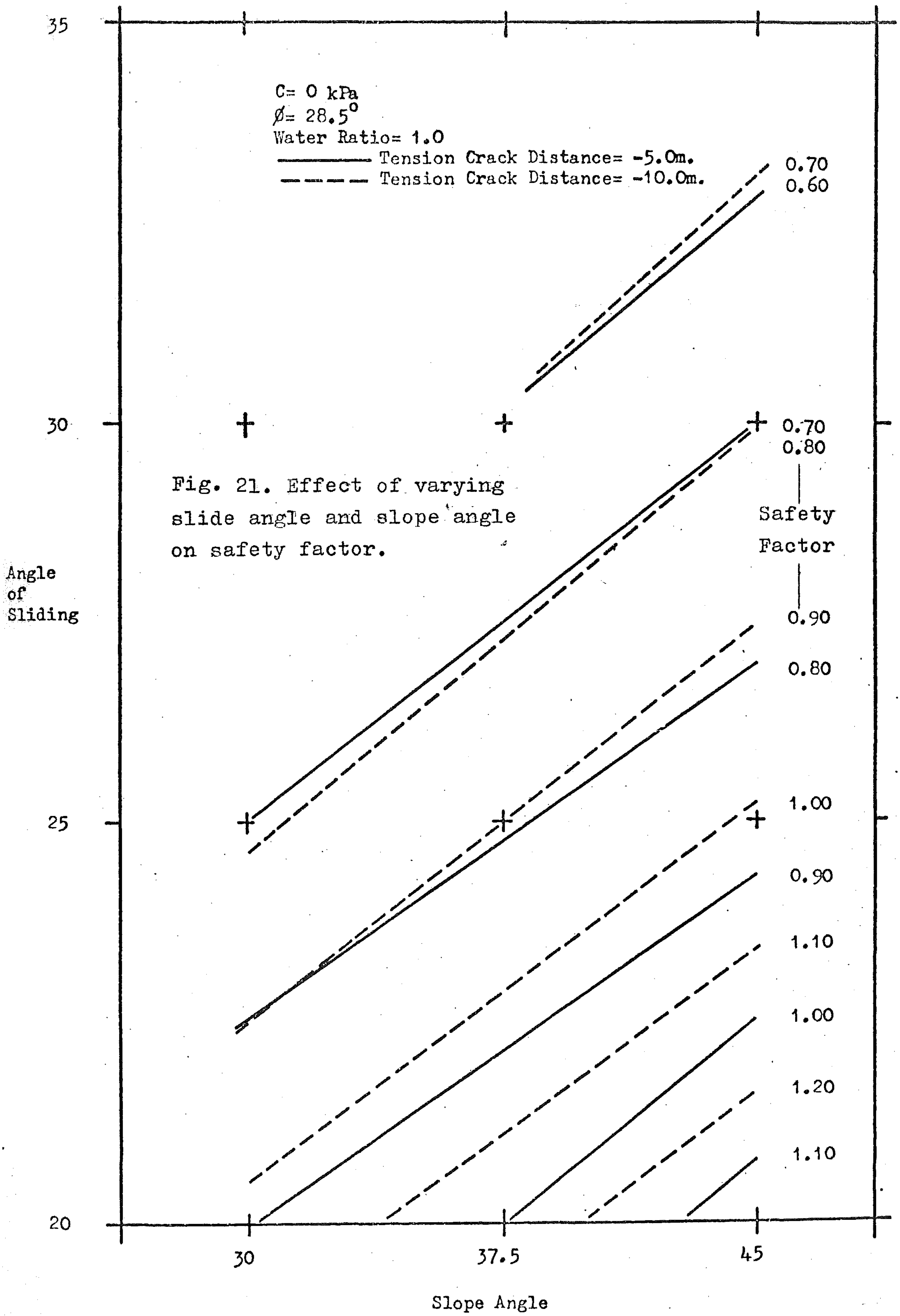


Fig. 21. Effect of varying slide angle and slope angle on safety factor.

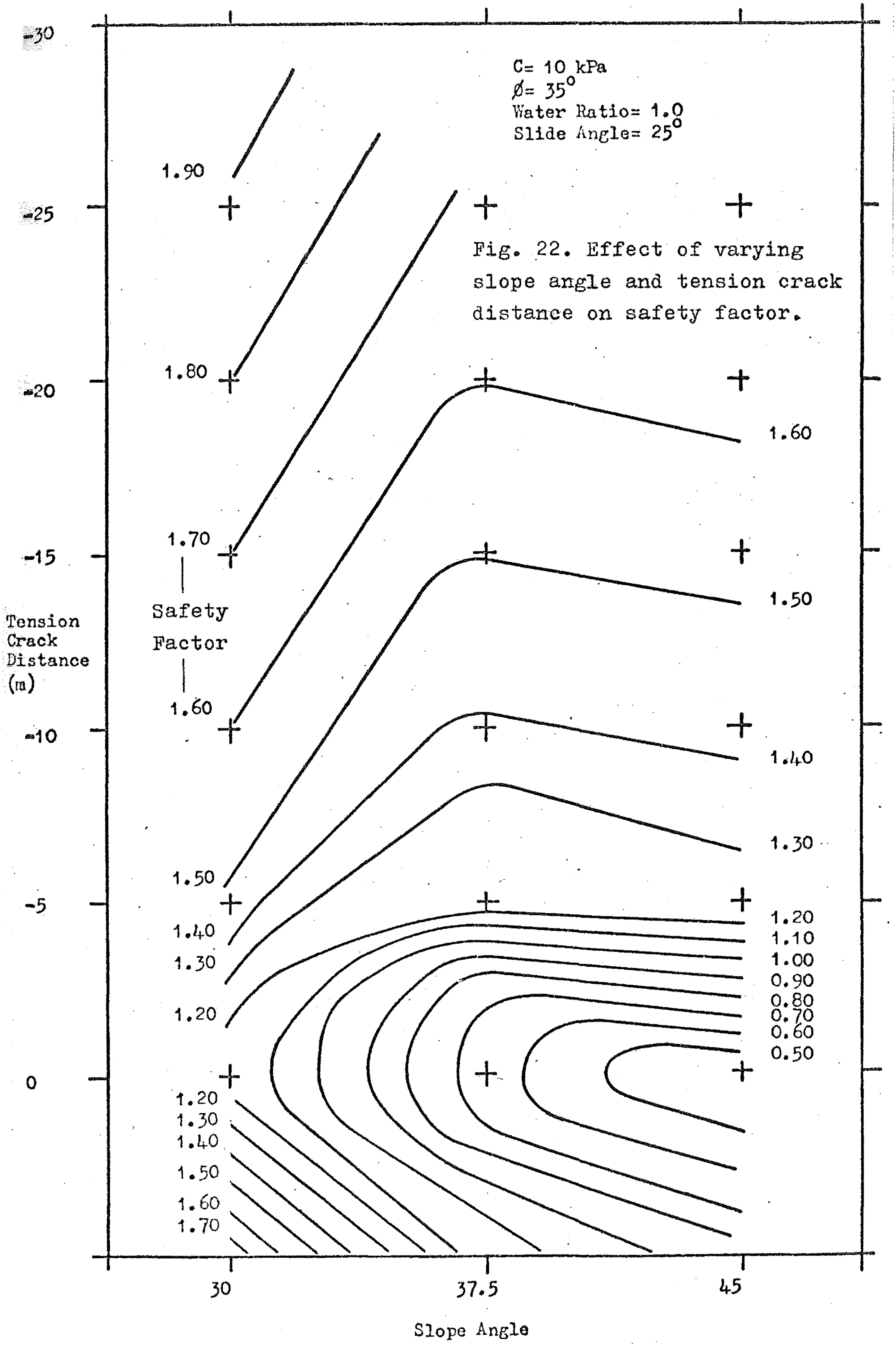


Fig. 22. Effect of varying slope angle and tension crack distance on safety factor.

3.7 CONCLUSIONS AND RECOMMENDATIONS

Slope instability at the Hawkswood Cut results in perennial track clearance and maintenance problems. The one kilometre long cutting was constructed in 1937 with batter angles of 45° and slopes rising to near 20m. Slides from both east and west slopes were recorded immediately following excavation and continue to the present.

The ground surfaces above the batter crests encourage ponding and runoff into the Cut. Two flumes and a cutoff trench provide inadequate surface drainage.

Soils through which the cutting is constructed include consolidated clayey silts and sandy silts, and weakly cemented gravels and cobbles. Fine grained soils most susceptible to instability under present slope angles are Beds C, I, J and K. Gravels and cobbles appear to stand competently under slope angles of 45°.

Instability is most prevalent in a 200m long central portion of the cutting. At this locality slopes are approaching the vertical. Outside this central section slopes attain a near vertical angle for 2-3m above slope toe, above which they generally rise at 45°.

The influence of water, either as runoff or in the creation of excess pore pressures, as a failure or triggering mechanism in slope instability cannot be over-emphasised. The association of increased landsliding with high intensity or prolonged rainfall has continued through 1976.

The favourable attitude of sedimentary bedding probably precludes large-scale structurally controlled earth movements in the future.

Laboratory tests reflect fine grained soils as clayey sandy silts approaching saturation under normal winter conditions, having low permeability and occasionally slaking heavily after exposure.

Soils tested under triaxial shear had angles of repose under steady seepage conditions slightly higher than 30^{o}. All soils were normally consolidated. Saturated normally consolidated soils under shear, as in an earthquake, especially silts and fine sands, are known to decrease voids inducing high positive pore water pressures.

Drill holes have confirmed the highly erratic distribution of gravels in the vicinity of the cutting. The use of gravels in a complex excavation design involving cut and benching, or their use as a natural medium in a subsurface drainage scheme, would therefore appear impractical.

Measurements of standpipe piezometers have revealed a complex hydrostatic regime involving a number of perched water tables. No conclusions relating an increase of piezometric pressure with short term high intensity rainfall or longer term seasonal variations have been drawn.

Railways are urged to make very effort to continue monitoring piezometers at regular intervals, preferably intervals of one week duration.

Theoretical slip circle slope stability analyses have shown stability to be dependent on both slope angle and pore water pressure. A small but measurable increase in safety factor will be achieved by reduction of a 45^{o} slope to one near 30^{o}; a similar margin of safety will be obtained if the initial 45^{o} slope is adequately drained. Substantial increases in safety factor will result if drainage is installed in conjunction with a slope reduction from 45^{o} to 30^{o}.

Plane failure stability analyses have reinforced pore pressure and slope angle as conditions affecting stability; other conditions affecting plane failure include changes in shear strength parameters, the location of tensional cracks in the slope and changes in angle of sliding.

Earthworks will be needed to reduce near-vertical slopes midway through the cutting to a more acceptable angle. If soils are adequately drained, a continuous, near-30^{o} slope, without benching, would appear to be acceptable. Widening of the present base level to a width which would allow the passage of earthmoving machinery alongside the railway should also be considered in the design of the new batters.

If earthworks outside this central locality are planned, the possibility of reactivation of rotational slumps described in section 3 should be considered. The complete removal of these features would alleviate any possibility of re-initiating movement.

It is recommended that the excavation be undertaken during the summer months to minimise wet weather construction problems. On completion of the construction, hydroseeding of the new batters should be undertaken as soon as possible. Soil and Water Division, M.W.D., are available for consultation on the most suitable grass type for the soil conditions at the site. In addition, on completion of the construction, consideration should be given to installing a slope-failure warning system. The system, consisting of a warning light sited at each approach to the cutting, would become activated should landsliding recur. Trains entering the cutting would therefore be forewarned of possible blockages and proceed with caution.

Surface drainage and ground surface treatment above the cutting must be considered in conjunction with any remedial earthwork scheme. Subsurface drainage may be considered as a last resort "brute force" measure. Drainage schemes are considered in section 5.

SECTION 4: THE MIKONUI EARTHFLOW - A TRANSLATIONAL, EARTHFLOW-TYPE LANDSLIDE

4.1 INTRODUCTION

Two kilometres south of Oaro Railway Station, the Main North Line traverses the toe of an extensive earth movement at a point known locally as Mikonui Point.* The landslide is referred to as the Mikonui earthflow. The principal southern Marlborough township, Kaikoura, lies 20 kilometres northeast of Mikonui Point (Fig. 23).

Following an existing topographic depression, the Mikonui earthflow extends inland from the coast at Mikonui Point 1300m, to a point 215m above sea level. The maximum active width, 200m measured parallel to the coast, tapers to 25-30m at the head of the landslide. Some 11 hectares are involved in slope movements. Two ridges, trending northeast, run parallel with but slightly back from the northwest and southeast boundaries of the earthflow. The lower (toe) boundary is defined by the coast, while at the head of the landslide a scarp separates earthflow debris from insitu (crown) material (Fig. 26). Width to length ratios vary from 0.15 at the toe to 0.02 at the head.

The south eastern boundary of the landslide is marked by a discreet zone of shear, the line of which can be traced almost continuously from the coast to the crown. A similar shear zone, from the head of the earthflow downslope to the confluence of Mikonui Stream with its

* Mikonui Point is not a recognised New Zealand Geographic Board locality.

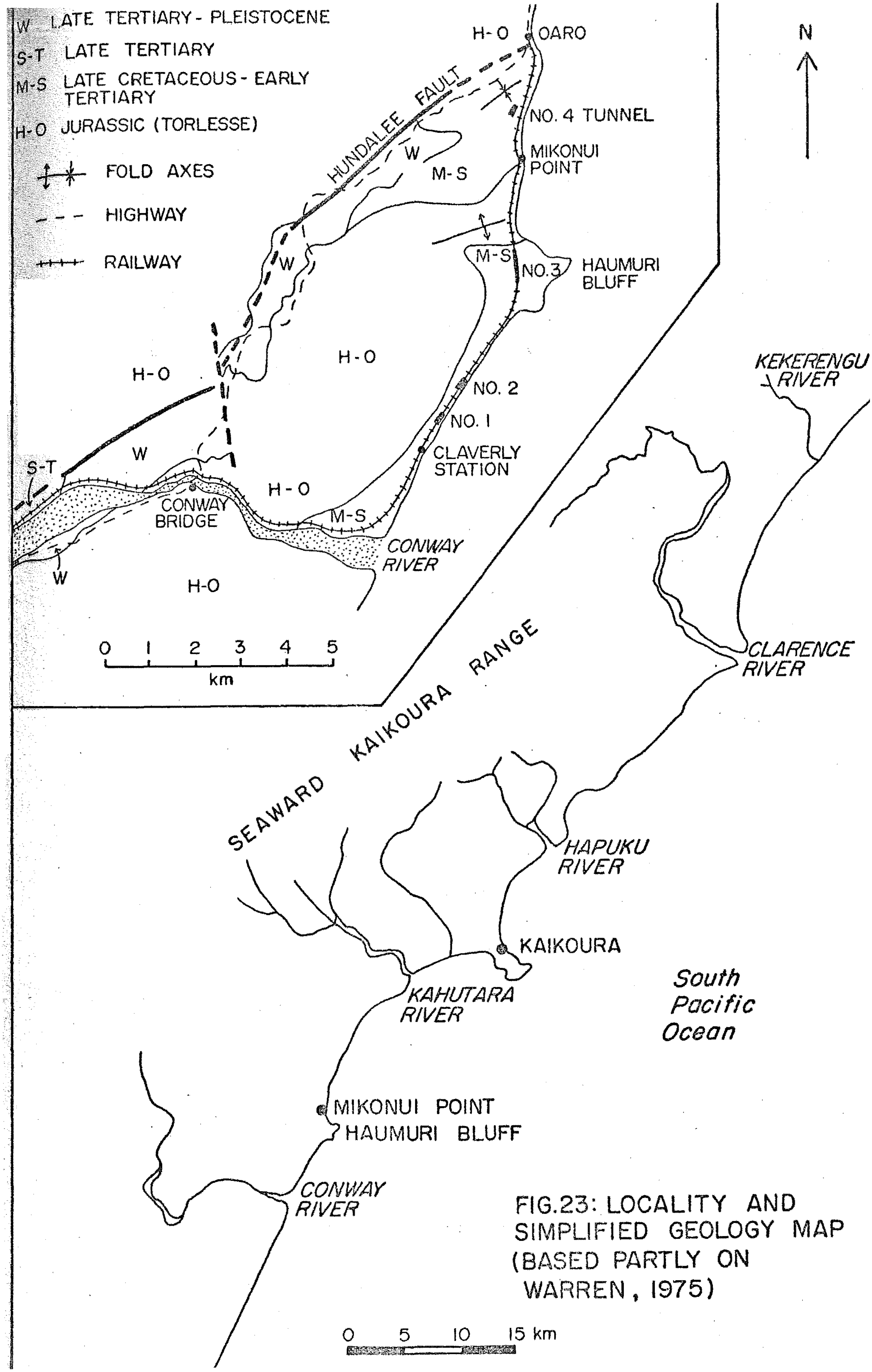


FIG.23: LOCALITY AND SIMPLIFIED GEOLOGY MAP (BASED PARTLY ON WARREN, 1975)

tributary, is observed or inferred as the north western boundary. Downslope of the confluence, the north western boundary follows the course of the Mikonui Stream to the coast.

The shear zone forming the lateral boundaries of the earthflow is a discreet crack, several centimetres wide at the ground surface, along which principally horizontal movement of the displaced soil occurs (Plate 18). However, in the upper (head) region of the landslide, horizontal and vertical displacements along the shear zone occur, the moving mass downthrown up to 1m; in this area the shear zone is clay-gouge-filled and highly slickensided. Commonly the shear zone dips steeply inwards, up to 70° from the horizontal, towards the centre of the earthflow.

At one locality in the southeast boundary, midway between the coast and the crown, reversed vertical displacement of the shear zone occurs. Here, the margin of the earthflow is upthrown 2m, producing a classical pressure ridge. The shear zone dips 60° away from the earthflow centre.

At several localities on the earthflow, or its environs, secondary landsliding occurs. The most significant locality is in the ridge adjacent to the southeast margin of the earthflow, at a height in the ridge equal with the crown (Fig. 26). Here, translational sliding of a large soil block towards the head of the earthflow occurs. This area is discussed in more detail in section 4.43. A further area of secondary slope activity occurs on the northwest edge of the earthflow between two points adjacent to the Mikonui Stream, 30-230m downstream from the confluence of the stream with its tributary. At this locality, slumping of a large soil block towards the stream is taking place. As a result of slope-lightening during excavation of surface drains in July, 1976 rotational-type slumping was initiated in slopes adjacent to the inland side of the railway. Following the excavation, a progressive development of tension cracks, and heaving near the track foundations, was observed.

The earthflow ground surface has typically gentle slopes, ranging 5^{o}-15^{o}. However, towards the centre of the landslide, slopes steepen to 25^{o}. Both ridges adjacent to the northwest and southeast boundaries of the earthflow are steep, with slopes typically 35^{o}-40^{o}. Near-vertical cliffs along part of the ridge adjacent to the northwest edge range to 100m in height.

The landslide ground surface has typically hummocky, crevassed topography, relief of 2-3m over short distances being common. In mid to upper regions of the earthflow, ponding in surface depressions, behind pressure ridges and at the rear of back-tilted scarps occurs.

The general appearance of the earthflow, a long, narrow tail diverging downslope to a wide tongue, flanked by high, steep sided ridges on both margins, has similarities to a glacial ice flow (Plate 16). As well, movement in earthflows of the type at Mikonui by a slow, creeping, plastic flowage of soil particles forming the slide mass is generally thought to take place; such displacements also have analogies to glacial movements. The mechanics of

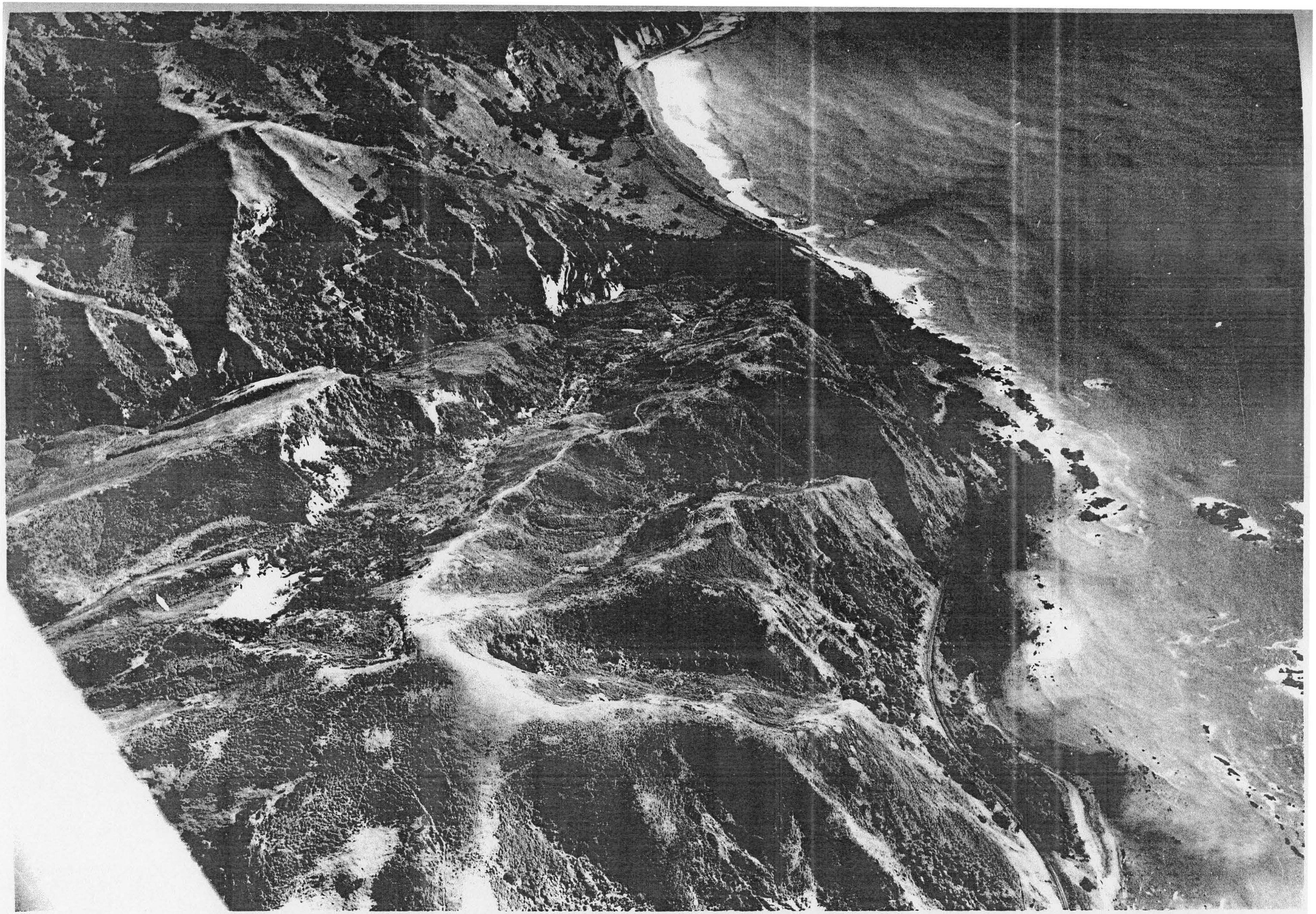


PLATE 16. The Mikonui earthflow, occupying the long, narrow depression in the c entre of photograph.

movement have not been a part of this study.

Creeping earthflows are notunique to the coastal Kaikoura region. Their occurrence are also reported from the East Coast, North Island (Olsen, 1974), where landslide characteristics similar to those at Mikonui, including a glacier-like shape, discreet lateral boundaries, hummocky topography, ponding and pressure ridges, are reported.

Continual maintenance of the railway at Mikonui Point results from a slow, continual, seaward-creeping motion of the earthflow. Such maintenance usually involves only track lifting and realignment every three to four months, though recurring movement during some years has necessitated realignment more frequently. Large displacements resulting in total disruption of the track do not occur. A 25kph speed restriction is currently enforced on rail traffic.

In 1974 a series of surface drains was constructed by N.Z. Railways in an attempt to drain the lower (foot) region of the earthflow. These were deepened and extended further up the slope during 1976. Most of the drains are aligned in the direction of movement and hence readily remove surface water during rainfall. Several of the drains are aligned across the slope and have an insufficient gradient to allow rapid runoff of water. The location of drains is clearly shown in Plate 18.

The following plan should be read in conjunction with section 4:

"The Mikonui earthflow - a site investigation plan".

PLATE 17. Surface drains in the lower foot region of the Mikonui earthflow. (Photograph by courtesy of the N.Z. Geological Survey).

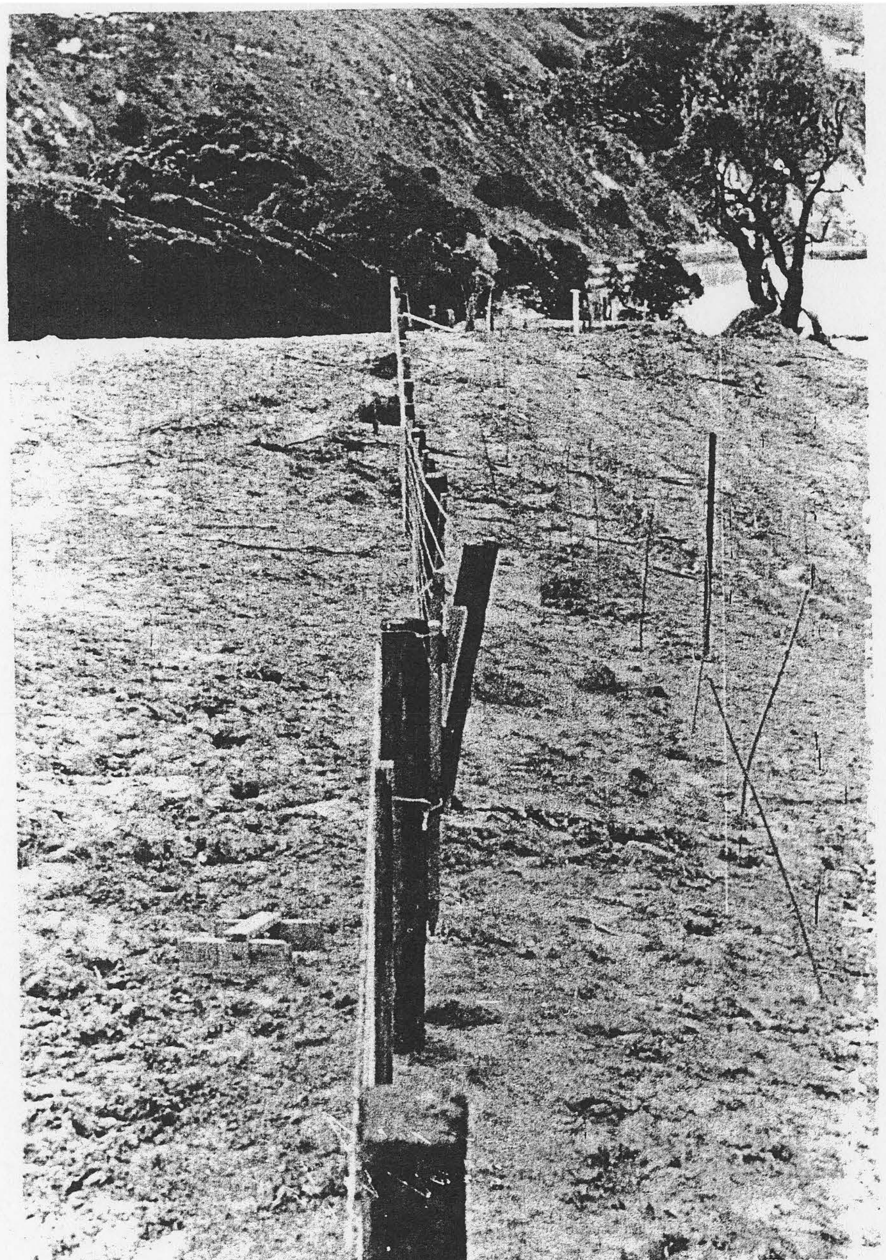


PLATE 19. Recent movement on the Mikonui earthflow illustrated by displaced fence post in centre of photograph.

PLATE 18. Shear crack forming discreet boundary of the landslide. Theodolite as scale.

4.2 METHODS

An initial reconnaissance of the landslide by sterographic viewing of vertical air photographs was undertaken.

run numbers	date
1797/65, 66, 67, 68	25.8.50
3947/24, 25, 26	20.4.66

Enlargements of both air photo runs of the area covered by the Mikonui earthflow were produced by N.Z. Aerial Mapping Limited, Hastings, at scales of 1:2000 and 1:5000.

Oblique aerial photographs of the site were also taken by the writer during September, 1976. A topographical base map of the landslide was executed at a scale of 1:1000 employing similar survey methods to those described in section 2.2. All spot heights were computer reduced which enabled simple plotting of 5m contour intervals on to a grid-coordinate base map. Mr. Wayne Lewis, assistant engineer, N.Z. Railways, assisted in the field. In addition, an unclosed pace and compass traverse of the Mikonui Stream and its tributary was performed.

Geological mapping of the site was carried out; this involved noting geological boundaries and descriptions of exposures. Geomorphic data, including slope angles, ponding, shear zones and tensional cracks, were also noted and added to the base plan.

Three boreholes in the foot region of the landslide were drilled for Railways to determine subsurface properties of the earthflow. An inclinometer access (slope deformation) tube was installed in one of the drill holes.

Ten survey marker stations were installed to measure rates of movement. Seven markers were installed in potential zones of activity, the other three outside the landslide. Rainfall monitoring and water table measurements were performed to attempt a correlation with rates of movement.

Strength testing of soil samples by Central Laboratories, Ministry of Works and Development, permitted a slope stability analysis of the Mikonui earthflow to be undertaken.

4.3 SITE GEOLOGY

4.31 Introduction

The Piripauan and Haumurian Stages of the New Zealand Mata Series (late Cretaceous) have their type locality at Haumuri Bluff, two kilometres southeast of Mikonui Point. Warren and Speden (1977 in prep.) provide the most recent accounts of the geology of the Haumuri Bluff area.

The following account of the geology of the Mikonui earthflow and its environs is based partly on the descriptions in Warren and Speden for the Haumuri Bluff district, and partly on geological mapping by the writer. Rocks outside, but adjacent to the landslide are described first, followed by an account of the material comprising the earthflow.

4.32 Stratigraphy

4.32.1 Rocks of the Earthflow Environs. The oldest rocks in the area comprise weakly metamorphosed greywacke sandstone and argillite (mudstone) of the Torlesse Supergroup. Characteristically hard and indurated, the rocks are complexly folded and in places highly fractured. Cannon-ball concretions of 0.2-0.4m diameter occur throughout the sequence. Typically sparsely fossiliferous, Torlesse rocks have a probable late Jurassic age in the Mikonui region.

Torlesse rocks are exposed at the coast, at and below sea level, to the fore of the landslide. The rocks continue south at sea level a further c. 1000m to the north portal of the No. 3 Main North Line tunnel. Inland, the contact separating Torlesse from overlying Cretaceous sediments follows approximately the line of the ridge adjacent to the south eastern earthflow boundary (see Site Plan).

As is the case regionally throughout New Zealand, the contact between Torlesse rocks and younger sediments is one of angular unconformity.

At two localities adjacent to the south eastern boundary of the earthflow, near the crown above the landslide, and at the railway, light blue-gray to dark gray sandstone is exposed. The rock is indurated when fresh, though in weathered sections is yellow-gray and heavily iron stained. With physical similarities to rocks of the Torlesse Supergroup, a Clarence Series (early-mid Cretaceous) age has been suggested for these sediments (Warren, pers. comm.). The rocks are of strictly limited distribution.

Rocks of the Piripauan Stage, Mata Series (late Cretaceous), (the Okarahia Sandstone), usually immediately overlie Torlesse sediments. In this study, rocks of the

Okarahia Sandstone have been subdivided into three members. The oldest Okarahia Sandstone member at Mikonui comprises a 4+m, fossiliferous, indurated, gray sandstone. Lenses of quartz and sandstone conglomerate occur throughout the rock. The sandstone is exposed at the coast, to the fore of the earthflow, and as deeply weathered sections in the ridge parallel to the southeast landslide boundary. The sandstone is thought not to be preserved at Haumuri Bluff.

Overlying the fossiliferous sandstone is a highly variable sequence comprising light bluish yellow siltstone, at least two montmorillonite clay (bentonite) beds and a dark reddish brown sandstone. The lower most siltstone is consolidated, slightly montmorillonitic and contains rare rhyolite pebbles. As well, blocks of silicified wood are included. Bentonites have a typical conchoidal fracture and waxy touch. The lower bentonite member is light bluish gray, with the basal part of the unit consisting of chocolate brown-dark gray-black, carbonaceous bentonite containing leaf fragments; the upper bentonite is light grayish green. The two bentonitic beds are separated by a loosely consolidated reddish brown sandstone. At the crest of the ridge paralleling the south eastern landslide boundary, adjacent to the head region of the earthflow, this bentonitic, middle member of the Okarahia Sandstone, attains a minimum stratigraphic thickness of 13m. As slumping at this locality within bentonites produces gravitational driving forces initiating slope movements in the earthflow (see section 4.43), a knowledge of the stratigraphy of this member of the Okarahia Sandstone has there-

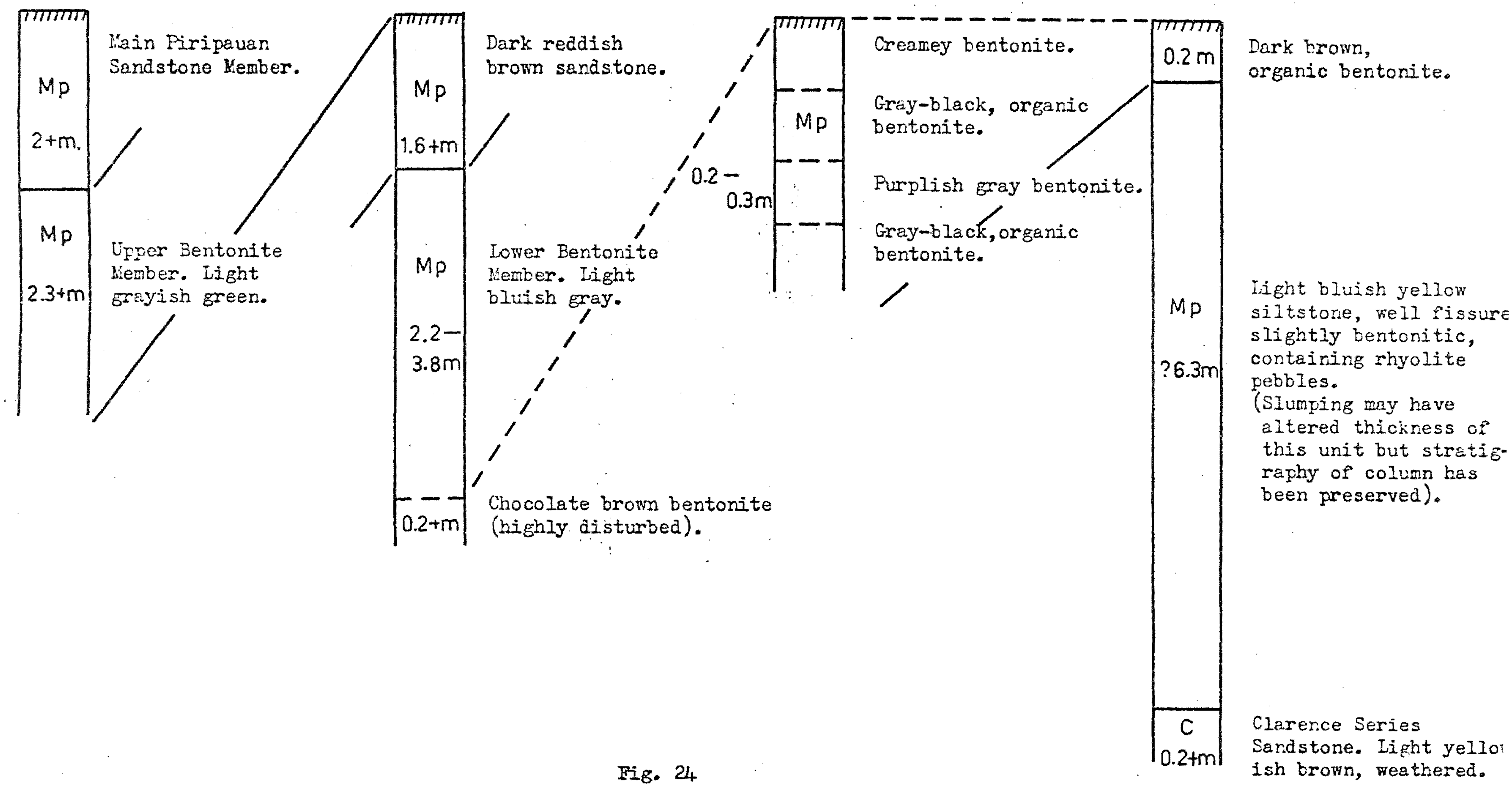


Fig. 24

Summary of critical Upper Cretaceous stratigraphy preserved at head of Mikonui Stream Tributary.

fore been critical. Figure 24 summarises the stratigraphy at the crown above the earthflow.

From drill hole data, montmorillonitic sediments are known to immediately underlie moving earthflow material in the lower half of the landslide. The soils comprise consolidated, blue gray-greenish gray, very pure bentonite, silt and sand with some montmorillonite clay, and a smooth fine to medium gravel conglomerate. In two of the three holes (A and C), the conglomerate forms a basal horizon, immediately overlying Torlesse Supergroup rocks. Borehole B (see Site Plan) recorded a 21.53+m stratigraphic thickness for these montmorillonitic sediments, though structural contouring of the basal conglomerate-Torlesse contact suggests a maximum 44m stratigraphic thickness in this hole. Bentonites, montimorillonitic silts and sands, and conglomerates underlying the earthflow are correlated with bentonites and montmorillonitic siltstones (Fig. 24) mapped at the crown above the landslide.

The youngest Okarahia Sandstone member comprises some 120m of interbedded, gray sandy siltstone, yellow-gray sandstone, gray and blue-gray siltstone and yellow-brown silty sandstone. A thin sandy conglomerate marks the base of this member. Bedding is typically 1-6m thick, occasionally cross bedded. Concretions to 1m diameter, fossiliferous bands and pyrites nodules may be present. The sediments are non-indurated. These siltstone-sandstone beds are referred to in this text as the main Okarahia Sandstone member, for at the Haumuri Bluff type section the sediments are possibly the oldest Piripauan rocks, resting directly on the Torlesse erosion surface (Warren and Speden). In the Mikonui area the rocks are seen as high cliffs and bluffs forming the ridge adjacent to the northwest earthflow boundary.

Conformably overlying the Okarahia Sandstone, up to 16m thick, is a brownish gray-mauve-dark green, fine-sand to fine-gravel conglomerate. The deposit may be calcite cemented, in which case it stands out at the coast as hard resistant strike reefs off shore of the toe of the earthflow; alternatively, the conglomerate may have only a friable strength. Formerly known as the Black Grit, the horizon is named in Warren and Speden as the Tarapuhi Grit. The rock has been assigned to the Haumurian Stage, Mata Series (late Cretaceous). Tarapuhi Grit forms a prominent dip slope to the northwest of the landslide (see Site Plan).

Warren and Speden infer marine deposition, near shore, for rocks of the main Okarahia Sandstone member. An earlier period of lacustrine or lagoonal conditions is suggested for the older bentonitic sediments. A shoreline environment is suggested for the Tarapuhi Grit.

Outside the region mapped in this study, siltstone (Conway Siltstone), sandstone and sandy limestone (Claverly Sandstone, Terodo Limestone) and limestone (Amuri Limestone), conformably overlie the Tarapuhi Grit. These sediments span the New Zealand Stages, Haumurian to Bortonian (late Cretaceous-mid Tertiary).

4.32.2 Soils of the Earthflow. The youngest sediments at the site are those within the earthflow itself, named the Mikonui Earthflow Formation. The sequence comprises a heterogeneous mixture of sandstone and silicified wood boulders and relic tree trunks, set in a matrix of sand, silt and bentonitic clay. Colluvium would be the usual term designated to such a deposit. Much of the colluvium is derived from rocks of the Okarahia Sandstone.

As a definitive age for the landslide is not known, a general Quaternary age has been assigned. Plant remains of Podocarpus totara/hallic (totara), Nothofagus sp. (beech) and Leptosperium scoparium (manuka) were kindly identified by B.M.J. Molloy, Botany Division, D.S.I.R. from borehole A at depths of 7.3 m and 13 m below ground surface. An interglacial climate is possibly suggested by these plants. A maximum 30.4 m thickness in drill hole B was recorded for the colluvium.

4.33 Structure

Post-depositional deformation of late Cretaceous-early Tertiary rocks in the Haumuri Bluff district resulted in relatively simple tilting of sediments about a set of northeast-southwest trending fold axes (Fig. 23). Between the fold axes, in the environs of the Mikonui earthflow, late Cretaceous rocks (Okarahia Sandstone) steepen to a 35^{o} dip towards the northwest.

4.4 FACTORS INDUCING EARTHFLOW MOVEMENTS

4.41 Geological Structure

The southeast and northwest lateral boundaries of the earthflow are flanked by two moderately steep, up to 200 m high, ridges, one on either side of the landslide. These ridges trend north eastwards, approximately coincident with the regional late Cretaceous-early Tertiary strike in the Haumuri Bluff district. The dip of sedimentary bedding in the vicinity of the earthflow has been shown to be towards the northwest at 35° (section 4.33). Late Cretaceous sediments forming the ridge adjacent to the southeast boundary of the landslide therefore dip towards the centre of the earthflow at an angle the same as, or slightly less than, the slope inclination (Fig. 25); the attitude of bedding within this ridge is therefore unfavourable to bedrock stability. Conversely, the dip of bedding in sediments forming the ridge adjacent to the northwest landslide boundary is away from the earthflow, an attitude favourable to bedrock stability.

The significance of geological structure is discussed further in subsection 4.43

4.42 Properties of the Clay Mineral Montmorillonite

The clay mineral montmorillonite is of particular engineering importance because of high swelling and compressibility characteristics and low shear strength. Chemically, montmorillonites consist of conjoining identical units made of an alumina octahedral sheet between two silica tetrahedral sheets (Krynine and Judd, 1957). Oxygen to oxygen inter-sheet bording is extremely weak; insertion of attracted water molecules causes swelling between sheets and weakening or disruptions of bonds.

The capacity for water uptake in montmorillonites is such (the moisture content of bentonites has been known to reach 400%), that significant volume expansions occur as clay water content increases. Alteration of the water

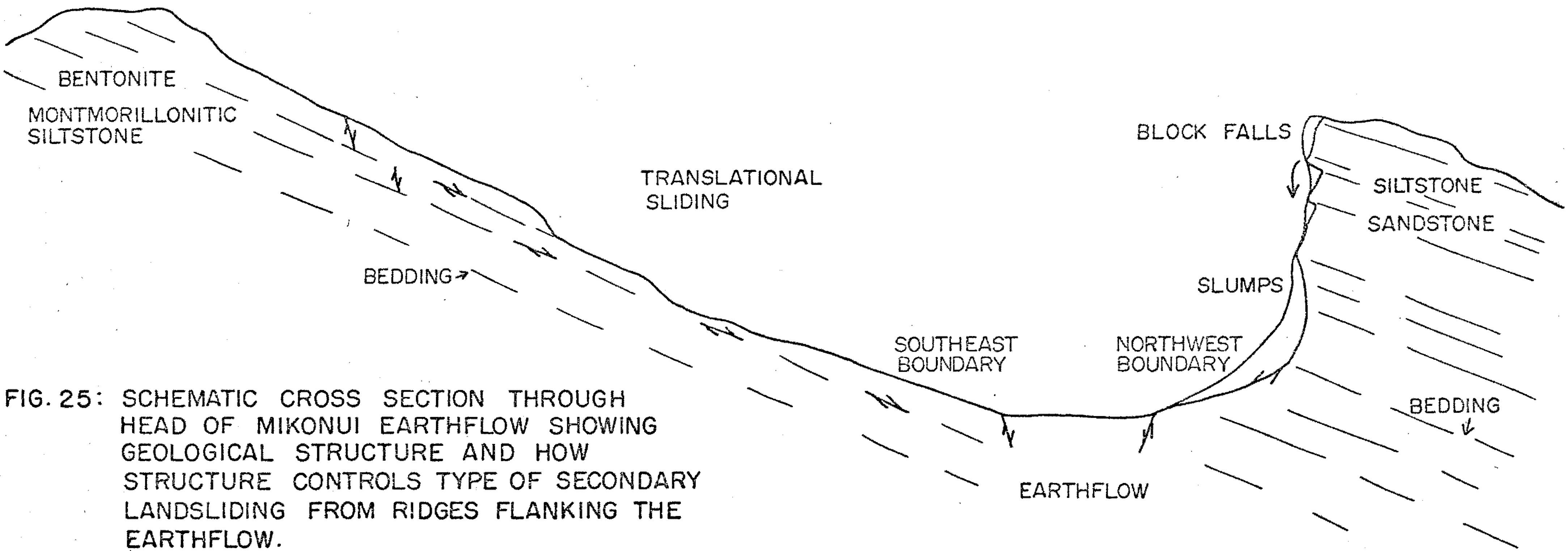


FIG. 25: SCHEMATIC CROSS SECTION THROUGH HEAD OF MIKONUI EARTHFLOW SHOWING GEOLOGICAL STRUCTURE AND HOW STRUCTURE CONTROLS TYPE OF SECONDARY LANDSLIDING FROM RIDGES FLANKING THE EARTHFLOW.

phase by the application or removal of load may result in similar volume changes (Fredlund, 1975). Shrinkage and cracking result when the clay mineral drys.

Low shear strengths are characteristic of montmorillonite expansive clays. This property may result from repeated disruption and weakening of inter-sheet bonding during swelling, or as a result of the parallel alignment of alumina and silica sheeting. Residual strengths of montmorillonites are typically the lowest of cohesive clays, with angles of internal friction 6^{o} - 14^{o} on average.

Physical properties of montmorillonite clays are dependent on the type of cations absorbed to the clay surface. The Mikonui bentonitic expansive clays are predominantly calcium montmorillonites (see section 4.6); swelling characteristics would likely be more severe and angles of internal friction lower had sodium been the absorbed cation.

Bentonite is the term for a particular, usually very pure, form of montmorillonite clay. Most bentonites originate from deposition of volcanic ash in marine or fresh water environments. If the deposition environment is marine, the clay is usually sodium rich; if fresh water, sodium depleted. Sodium depleted bentonites at Mikonui overlie slightly montmorillonitic siltstones containing rhyolite pebbles. Thus, deposition of an acid volcanic ash under fresh water conditions is suggested for the Mikonui bentonites.

Low shear strength and high swelling properties lend montmorillonite clays to be particularly susceptable to landsliding. The significance of landsliding in bentonites in the Mikonui region is discussed in the following subsection.

4.43 Zones of Active and Passive Slope Movements

An area of secondary landsliding at the crest of the ridge adjacent to the south eastern boundary of the earthflow was discussed in section 4.1. At this locality (Fig.26), downslope movement of a soil block, of one hectare area, towards the head region of the earthflow occurs. The south and west boundaries of the moving block are marked by a discreet scarp; a slickensided shear zone defines the east perimeter. Sediments within the displaced block comprise bentonites and montmorillonitic siltstones of the Okarahia Sandstone (Fig. 24), described in section 4.32.1

Slope failure at this locality within bentonitic soils is attributed firstly to the low shear strength of the sediment, due to the presence of the clay mineral montmorillonite, and secondly to the unfavourable attitude of bedding, a dip towards the centre of the earthflow (section 4.41). Displacement of the soils towards the head of the earthflow takes place by translational sliding along and down bedding planes.

In addition to the deposition of bentonitic soils, an accumulation of debris at the head of the earthflow results from block falls and slumps within sandstone-siltstone sediments forming the ridge adjacent to the northwest landslide boundary. Block falls and slumps occur in this ridge from the crown, downslope to a point two hundred

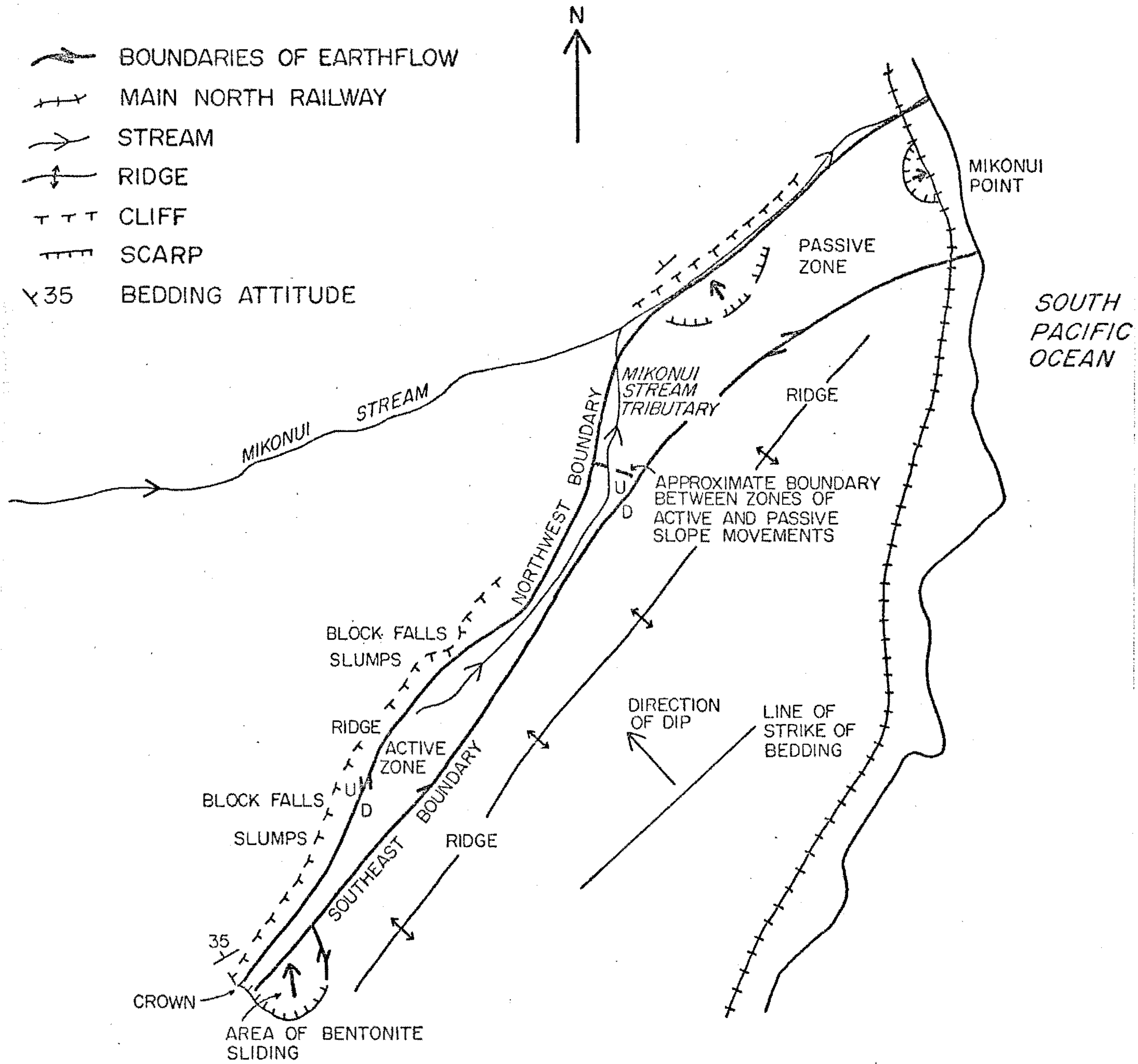


FIG. 26: GEOLOGICAL STRUCTURE IN RELATION TO TREND OF EARTHFLOW AND LOCATION OF AREAS OF SECONDARY LANDSLIDING.

metres above the confluence of Mikonui Stream with its tributary. The volume of material contributed to the earthflow through falls and slumps exceeds that derived from the opposite ridge, though the quantity is not known.

On the basis of zones or areas of material accumulation, the earthflow has been subdivided into two sections of contrasting slope activity. The upper (head) earthflow area, designated the zone of active slope movements, contrasts with areas lower in the slope where no secondary landsliding from ridges adjacent to the earthflow result in sediment deposition into the landslide (Fig. 26). The lower (foot) area is designated the zone of passive slope movements. Downslope, gravitational forces within the zone of active slope movements, caused by the weight of continuing deposition of debris, provide the essential driving forces controlling movements lower in the slope. In both active and passive zones, however, actual rates of movement are possibly controlled by changes in soil pore water pressure. Should remedial measures be contemplated at the site, they must be planned with a view to either decreasing the driving forces in the zone of active slope movements, or increasing the shearing resistance of the zone of passive slope movements.

Translational dip slippage within bentonitic sediments has probably not always been restricted to the present area of landsliding in the ridge adjacent to the southeast earthflow boundary. Possibly the head of the earthflow has reached its present position by upslope migration along a pre-existing topographic depression; landsliding within bentonites at successive head levels in the earthflow would probably have accompanied this upslope migration.

4.5 FIELD INVESTIGATIONS

4.51 Cable Tool Percussion and Diamond Drilling

Three boreholes were drilled for Railways by the Christchurch based Investigations Section, Ministry of Works and Development. Borehole locations, shown on the Site Plan, were all within the lower (toe) region of the earthflow.

Two truck-mounted drilling rigs, a Pioneer rotary and a Cyclone cable tool, were used. The rotary was equipped with NQ size, split-inner, triple-tube core barrels; wire line core recovery methods were employed. A monkey of weight 611 kg (12 cwt) dropped from 76.2 cm (30 in.) was used in conjunction with 15.4 cm (6in.) I.D. casing on the cable took rig; stationary thinwall open end piston samplers and Raymond spoon samplers were used to obtain soil samples.

Drilling was undertaken between early April and mid July, 1976. Initially, all holes were to be drilled by the cable tool method. Unacceptably long time delays during boring of hole A resulted in the percussion rig being withdrawn and the hole left cased before end-of-hole had been reached. An available rotary rig was brought in to finish the hole. A similar technique, commencing the hole using percussion methods and finishing with rotary, was employed in holes B and C.

The following is a summary of the drilling methods:

	Cable Tool	Rotary (Fish tail bit)	Rotary (Continuous core)
hole A	15.92m:		-7.86m:
	-7.86m:		-12.42m:
hole B	23.6m:		12.6m:
	12.6m:		-33.09m:
hole C	41.98m:	31.32m:	17.91m:
	31.32m:	17.91m:	-5.87m:

Drilling was undertaken in order that the following information could be obtained:

(a) Depths to recognisable sliding surface(s), if any.

(b) Correlation of water table levels with rainfall and ultimately rates of movement.

(c) Effect of future remedial drainage, if any, on hydrostatic levels in boreholes.

(d) Strength and index testing of soil samples, and hence slope stability analyses.

(e) Borehole correlation with proposed seismic refraction traverses across the earthflow, if geological conditions suit.

(f) Deformation of boreholes at depth and correlation with movement.

With the exception of hole B, all holes were drilled until basement rock (Torlesse Supergroup sandstone) was penetrated. The decision by the writer to abandon hole B before basement was reached appears justified; geological structural contouring inferred an additional 84m drilling from end-of-hole until basement was penetrated. Little additional information could have been gained.

Conclusions from the drilling programme include:

(a) Although an actual slip surface was not recognised due to a lack of recovered drill cores at the critical depths, a zone of shear on and above which movement in the earthflow is occurring has been inferred.

(b) The boundary between earthflow and underlying, apparently insitu, material was recognised by changes in soil lithology. Material above the inferred failure surface is a colluvium, comprising a heterogeneous mixture of sandstone boulders and organic material (including tree trunks), in a matrix of sand, silt and bentonitic clay. Colluvium is referred to in section 4.32 as the Mikonui Earthflow Formation. Sediments immediately underlying the inferred zone of shear comprise relatively dense, firm, dry bentonite, montmorillonitic silts and sands, and conglomerates of the Okarahia Sandstone. Carbonaceous fragments occur within fissures throughout these deposits. Bentonites typically show no features indicative of deformation, such as disruption of fissures and general loss of soil strength. The Okarahia Sandstone in turn overlies hard, indurated sandstones of the Torlesse Supergroup.

(c) The depth of the inferred shear zone below ground surface ranges from 15m and 20m in boreholes A and C, to 30.4m in hole B. With respect to sea level, the failure surface occurs less than 1m above in hole A, 6.8m below in hole B and 22m above in hole C.

(d) Three dimensional analysis of the inferred failure surface indicate the shear zone has an apparent 10° dip in the direction of sliding. The true dip is 14° towards slightly west of north.

(e) In the area of the earthflow covered in the drilling programme, the inferred failure surface has a planar shape in the direction of sliding and is approximately parallel to the ground surface. Movements are therefore of a translational nature.

(f) The firm, dry, dense nature of the bentonitic sediments indicates that soils beneath the colluvium-bentonite interface are effectively impermeable. Pore waters within the colluvium will therefore be perched above this interface. As well, erratic grain size variations in the colluvium matrix with depth suggest pore waters are likely to be perched within those colluvial horizons of higher permeability.

(g) Insufficient density contrast between colluvium and bentonite would prevent seismic refraction techniques being employed in the calculation of depths to the failure surface.

Drill core logs are given as appendix 4. Core recovery within sediments of the Okarahia Sandstone was generally excellent, over 95% recovery typical. This was due mainly to the dry, firm, dense nature of the soil. Less impressive was the 40-50% average core loss experienced while drilling through the colluvium, mainly due to erratic changes in grain size and the disturbed nature of the material.

4.52 Borehole Deformation Studies

The installation of inclinometer access tubing in earth movements is undertaken for the location and orientation of failure surfaces at depth within a landslide. These were the principle reasons for installing access tubing in the Mikonui earthflow; however, monitoring of inclinometer tubing clarified a significant geotechnical problem which remained unanswered following completion of the drilling programme. Drill hole data suggested only sediments of the Mikonui Earthflow Formation (colluvium) are involved in the movement. However, due to the low shear strength and susceptibility to landsliding of montmorillonite clays, the possibility of bentonites underlying the colluvium being involved in the movement had to be resolved. If bentonitic soils are being displaced, the most likely location of a failure surface would be at or near the lithology boundary with rocks of the Torlesse Supergroup.

Borehole inclinometer access tubing (slope deformation tubing) was installed in borehole C. The tubing consists of 3m lengths of 5cm ID circular aluminium with four equi-spaced keyways or guide grooves running down each length (Fig. 31). Lengths of tubing are coupled by means of riveted aluminium sleeves. A 50mm gap between tubing ends within each sleeve allows the instrument operator to locate couplings. An end cap and sealing tape round all couplings prevents the ingress of grout during installation.

Inclinometer tubing in borehole C was installed in a 7.62cm diameter hole after the initial NQ size hole had been reamed with NW casing. The hole was filled with water and 6m lengths of tubing lowered at each time. Clean water placed inside the tubing allowed buoyancy effects to be overcome. After installation, the tubing was rotated to orientate one set of keyways in the direction of expected maximum movement. The space between the sides of the borehole and access tubing was filled with a weak grout comprising the following mix; in proportions by weight:

cement	bentonite	fine sand	water
4	1	18	8

The grout mix was supplied by Central Laboratories, Ministry of Works and Development. Finally, an instrument working platform consisting of a square concrete cap was placed round the tubing at the ground surface.

The inclinometer instrument comprises a strain transducer torpedo of length 25.4cm (supplied by N.Z. Geological Survey) having two fixed and two spring-loaded wheels attached to a length of graduated cable and a digital-strain indicator. The strain transducer measures the angle with the vertical of the tube; horizontal displacements are calculated by changes in the vertical angle with time.

Recordings of the strain transducer are taken from the bottom of the hole upwards at specified vertical intervals (stations). Both forwward and reverse readings for each pair of keyway axes are taken (N,E = forward; S,W = reverse). The increase in the vertical angle in the forward keyway will be the same as the decrease in the reverse. Readings across couplings are to be avoided.

Inclinometer access tubing was placed the length of hole C (47.85,), the bottom 3.02m of which penetrated basement rock of the Torlesse Supergroup. Thus, the basal lengths of access tubing were known definitely to be in unyielding material. A discrepancy arose between the actual length of tubing installed and the length of tubing as measured by the inclinometer torpedo cable (46.63m). Errors in the graduated cable length may account for this.

Figure 27 illustrates results of inclinometer access tube monitoring in the direction of the long axis (maximum displacement) of the earthflow in borehole C. The base station was established on 27 July, 1976. Three subsequent recordings were possible; at the time of the fourth measurement (30 September), the access tube had been sheared by earth displacements.

Lithology changes occur at the following inclinometer graduated cable depths below ground surface:

Colluvium - bentonitic soils (Okarahia Sandstone) :
66 ft (20.1m)

Bentonitic soils - Torlesse Supergroup :
143 ft (43.6m).

Conclusions from the inclinometer monitoring programme include:

(a) A single basal shear zone was located at a depth coincident with the colluvium - bentonite interface. No other zones of shear were located, either above or below, this boundary. Thus, bentonitic soils of the Okarahia Sandstone are insitu and form the surface above which sliding of the colluvium occurs.

(b) Inclinometer measurements in the direction of the landslide long axis indicate the expected downslope (northeast) direction of movement in the earthflow. While monitoring lasted, colluvium was relatively displaced a maximum 6.9cm.

(c) Inclinometer measurements in the direction of the

OVERLAY TO ACCOMPANY FIG. 27

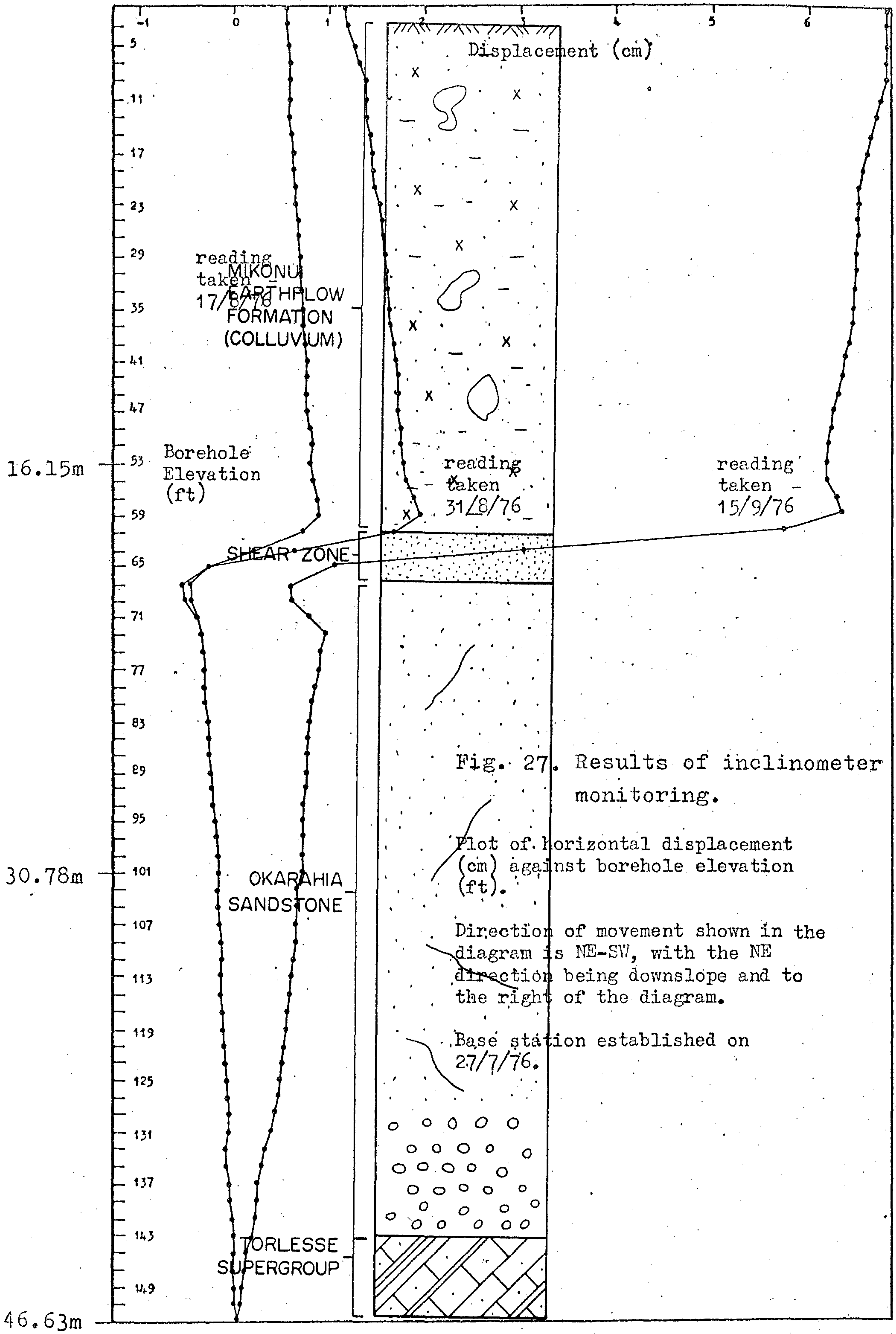


Fig. 27. Results of inclinometer monitoring.

Plot of horizontal displacement (cm) against borehole elevation (ft).

Direction of movement shown in the diagram is NE-SW, with the NE direction being downslope and to the right of the diagram.

Base station established on 27/7/76.

short axis (across the earthflow) indicate a similar, albeit much smaller, horizontal displacement towards the southeast boundary of the landslide.

(d) The zone of shear extends from depths 60-66 feet (18.3-20.1m) below the ground surface.

(e) Above the shear zone, displaced material within the earthflow appears to be undergoing deformation at an approximately constant rate. Rates of deformation are compared with surface displacement of markers, discussed in 4.45.

(f) Below the shear zone, the orientation of the access tubing remained constant for the first two (17.8.76, 31.8.76) readings. The final reading (15.9.76) indicated a 1cm horizontal difference with the previous two readings at a point measured immediately below the failure plane. This displacement of the tubing below the shear zone, in the writer's opinion, represents collapse of the enclosing grout mix due to shear stresses placed on the tubing in the earthflow immediately overlying, and does not indicate actual movement within bentonitic sediments.

(g) Perforated PVC tubing, installed in borehole B on completion of the drilling programme to measure water tables, became sheared at a depth 14.9m below the ground surface two months after being installed. This depth is again coincident with the colluvium-bentonite interface.

4.53 Rainfall Monitoring

A Marguis 600 Series rainfall gauge was installed at the site in April, 1976. The gauge is read daily by the district Railways Inspecting Ganger. A New Zealand

Meteorological Service recording station number, H23551 (Mikonui) has been assigned to the gauge. As well, a Meteorological Service recording station H23641 (Conway Flat) has been recording rainfall from a coastal district 15km to the south since 1950 (Fig. 3).

Table 11 summarises rainfall for these stations. During the time covered by this study, the wettest months at the site were July, 1976 and January, 1977. These were followed closely by October and December, 1976. As with rainfall at Ethelton, the months leading up to and including June, 1976, are dryer, while those following June are consistently wetter, than the district norm. No months experienced exceptionally heavy rainfall.

4.54 Water Table Measurements

In boreholes A and B perforated PVC tubing of internal diameter 5.1cm (2 in.) was installed after completion of the drilling programme to record water levels. The space between the sides of the borehole and the PVC was backfilled with a coarse sand/fine gravel (pea-gravel) filter. As the tubes are not sealed at pre-selected levels down each borehole, they record only average water tables and not true piezometric pressures.

In addition, water tables in borehole C have been recorded by measurements in the inclinometer access tube.

Water table levels were recorded using a simple battery-operated electrical probe coupled to a voltmeter on the end of a length of wire coil. Results of measurements are given in Table 12.

A plot of water table levels versus rainfall is

TABLE 11

MONTHLY RAINFALL FOR STATION CONWAY FLAT (1950-75), AND WEEKLY RAINFALL FOR STATION MIKONUI (APRIL 1976-FEBRUARY 1977)

January	67	198	6	49	42	15	78	0	135
February	62	123	23	112	0	0	23	5	58
March	73	215	11	21					
April	86	253	16	37	0	26	0	7	33
May	95	331	16	55	3	39	1	3	46
June	62	204	9	41	3	0	39	7	49
July	77	234	17	100	3	18	112	9	142
August	76	215	3	54	20	12	11	42	85
September	43	201	6	58	21	57	5	8	91
October	61	178	13	69	8	46	52	10	116
November	61	179	4	48	21	21	0	33	75
December	68	209	15	84	41	11	25	29	106
Total	832	1207	400	728					

1 = mean, 1950-1975

2 = high, 1950-1975

3 = low, 1950-1975

4 = 1976 figures

TABLE 12

SUMMARY OF WATER TABLE LEVELS, MIKONUI EARTHFLOW,

IN DEPTHS BELOW GROUND SURFACE

Date	DH.A (m)	DH.B (m)	DH.C (m)
10. 6.76	9.2		
16. 6.76	10.7		
24. 6.76	10.7		
8. 7.76	10.99	14.71	
14. 7.76			
21. 7.76	10.6	15.0	
3. 8.76	10.84	15.24	2.53
17. 8.76			2.91
31. 8.76	10.6	BLOCKED	3.05
10. 9.76	10.48	15.17	3.08
17. 9.76	10.78	BLOCKED	3.07
8.10.76	10.75		3.17
15.10.76	10.42		3.19
22.10.76	10.14		3.27
13.11.76	11.4		3.24
19.11.76	10.3		3.26
26.11.76	8.43		3.45
3.12.76	8.28		3.39
10.12.76	8.78		3.37
17.12.76	8.2		3.22
24.12.76	9.2		3.18
7. 1.77	10.0		3.40
27. 1.77	10.28		3.33

shown in Fig. 28. A correlation of rainfall with water table levels is not obvious. This may partly be due to the fact that boreholes are recording only average water table levels, and not true piezometric pressures; alternatively, a time-delay effect may be in operation, in which case a longer period of recording will be needed to correlate precipitation with water table levels on a satistical basis. However, this negative result follows a similar trend to measurements undertaken at the site of the new Poro-o-Tarao Tunnel, North Island Main Trunk Railway. At this locality, standpipe piezometers record no obvious rise following heavy rainfall (G. Borrie, pers. comm.). More sophisticated electrical piezometers (vibrating wire) may be needed to effect a correlation of rainfall with hydrostatic pressure.

4.55 Movement of Surface Markers

Ten surface survey markers were installed during March, 1976, the locations of which are shown on the Site Plan. Seven of the markers were located in the lower (toe) earthflow region, the other three outside the active zone.

The survey instruments and methods used in monitoring the markers are similar to those described in section 2.61. The initial survey performed by the 3rd Professional Class, School of Engineering, University of Canterbury, was undertaken during 3 and 4 April, 1976. The results of all resurveys have been compared with the coordinates and reduced levels obtained from the initial survey.

Slope distances and bearings measured during the initial survey were recorded in the following manner:

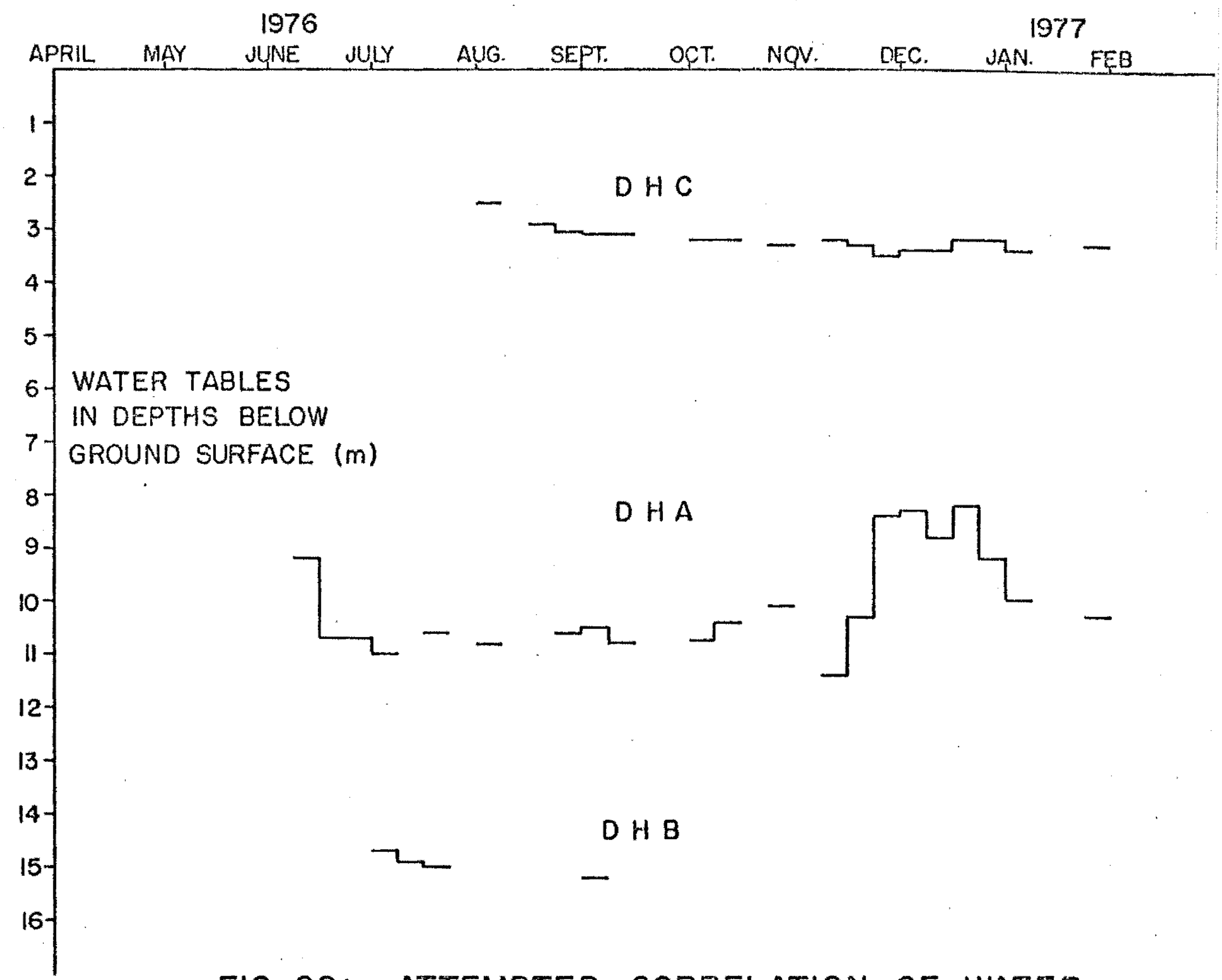
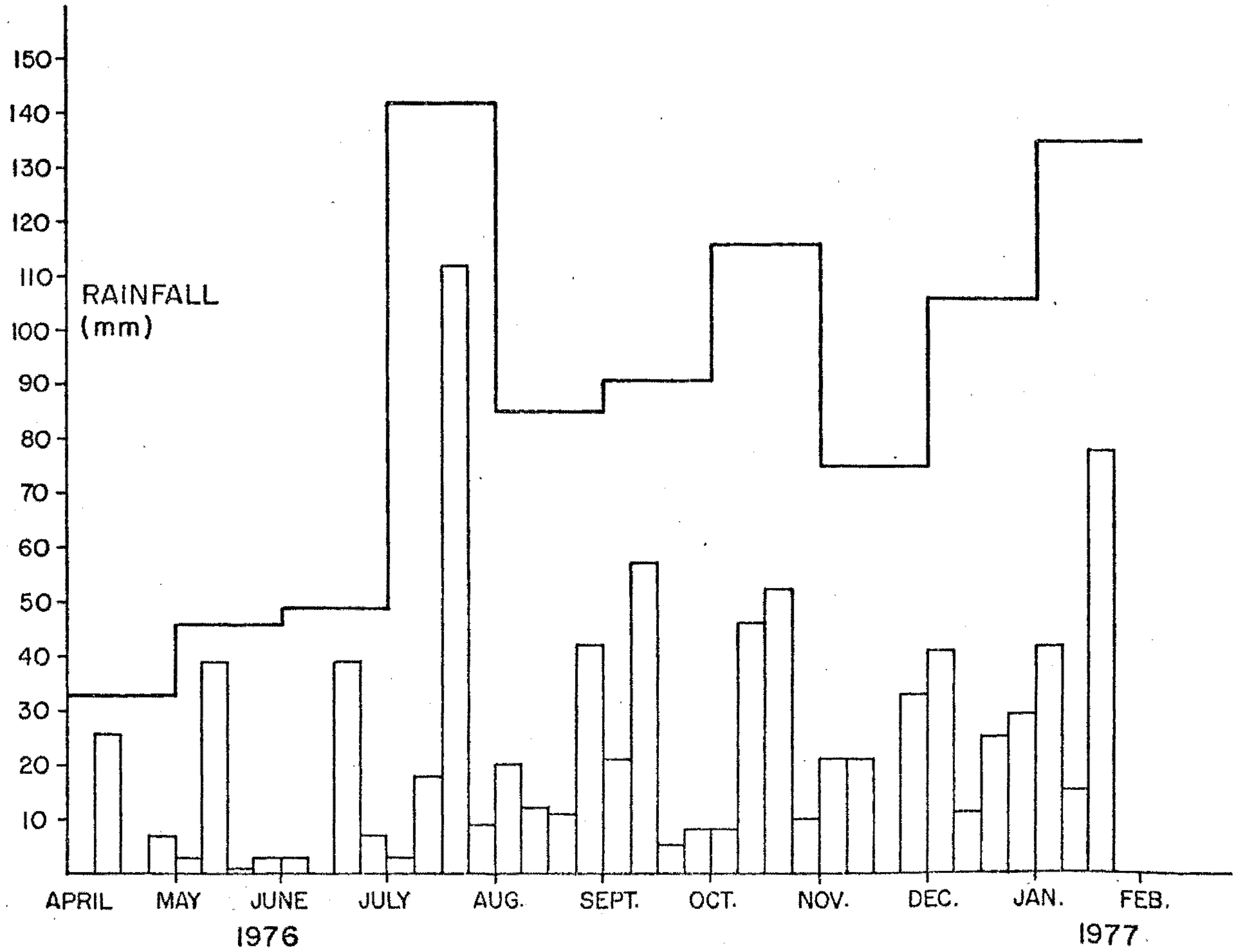


FIG. 28: ATTEMPTED CORRELATION OF WATER TABLE LEVELS WITH PRECIPITATION



from station no.	to station no.
1	5,8
2	3,5,8
3	2,4,5,8
4	3,5,8
5	1,8
6	8
7	8
8	1,5
9	8
10	8

During the re-surveys performed by the writer, the following methods were used:

from station no.	to station no.
8	2,3,4,6,7,9,10 bearing, vertical angle, slope distance
8	1,5 bearing, vertical angle

In addition, surveying of the inclinometer access tube was commenced after its installation in borehole C. Station 6 had to be re-sited after the first re-survey following its accidental removal.

As with the Ethelton monitoring programme, no absolute check on the stability of fixed stations 1, 5 and 8 can be assured during the re-surveys performed by the writer. However, as the bearing difference between stations 1 and 5 from control station 8 remained constant throughout the time of this study, the stability of the markers has remained unquestioned. A similar 20mm accuracy has been applied as the allowable error.

Table 13 summarises computer calculated results of precise surveying of surface markers.

The following conclusions concerning precise surveying of surface markers have been drawn:

(a) All stations located on the earthflow, except marker No. 2, have shown significant movement since the initial survey of 4.4.76. Within the 10 month period of this study, the magnitude of this movement has ranged between 13.2cm on station 4 to 91.1cm on station 6.

(b) The magnitude of movements appear consistent with previous N.Z. Railway's field observations that annual displacements in the toe region of the earthflow of 0.5-1.0m are usual.

(c) The direction of movements (Fig. 29) are consistent with a downslope displacement of the earthflow towards the coast. All markers upslope of the track experienced horizontal displacements towards the coast, as well as reductions in reduced level (maximum 14.7cm on station 10); displaced markers between the track and coast indicate only significant horizontal movements.

(d) The maximum magnitude of displacement, 91.1cm, experienced on station 6, is partly attributable to secondary slope activity (slumping accompanying tension cracking) following slope treatment in the vicinity of the marker during construction of surface drains in July, 1976.

(e) Monitoring of surface markers indicates movement on the earthflow has analogies to an ice glacier in that displacements are slow, but continuous; displacements at the

TABLE 13

RESULTS OF PRECISE SURVEYING OF MARKER STATIONS

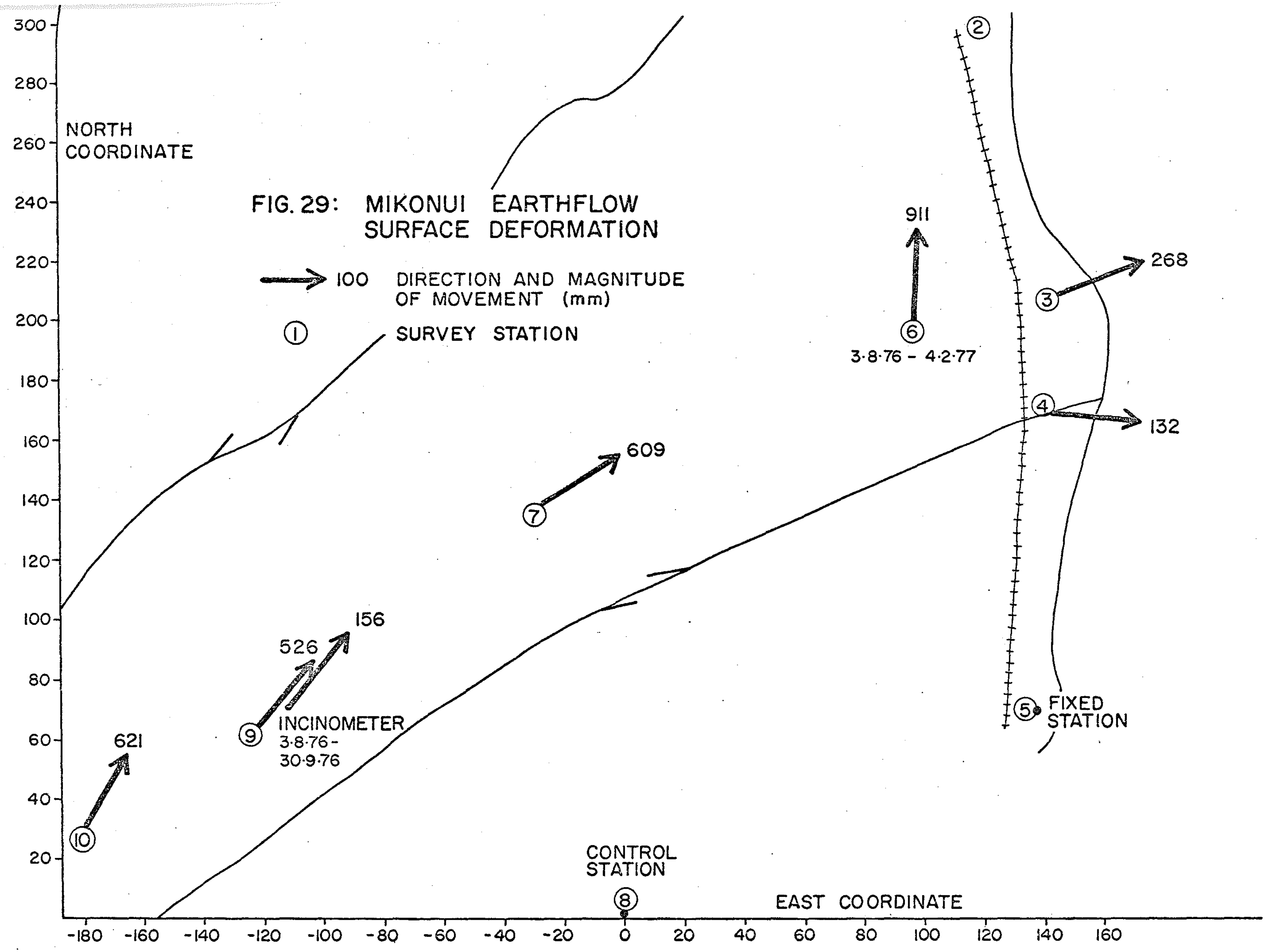
AT THE MIKONUI EARTHFLOW

ALL READINGS TAKEN FROM FIXED STATION NO. 8

station		29.5.76	3.8.76	30.9.76	4.2.77
2	a	00.03	00.00	00.03	00.10
	b	-11	0	-11	-10
	c	25	34	28	35
	d	29	34	32	41
	e	15.8	2.3	0	1.9
3	a	00.04	00.23	00.51	02.06
	b	7	48	87	206
	c	4	7	3	-2
	d	21	68	119	268
	e	11.5	21.4	26.4	35.2
4	a	00.24	00.07	00.25	01.34
	b	-12	-8	1	44
	c	-40	-47	-49	-54
	d	49	50	60	132
	e	26.7	0.5	5.2	17.0
6	a	00.49		03.37	11.32
	b	12		201	506
	c	-35	new	-50	-143
	d	63	origin	311	911
	e	34.4		160.9	215.2
7	a	00.31	01.32	05.43	13.49
	b	10	27	85	214
	c	-42	-40	-54	-71
	d	48	81	255	609
	e	26.2	15.0	90.0	83.6
9	a	00.40	01.22	05.08	12.40
	b	-11	-20	-61	137
	c	-44	-43	-55	-78
	d	53	73	223	526
	e	28.9	9.1	77.6	71.6
10	a	00.37	01.11	04.16	10.32
	b	-16	-28	-96	220
	c	-49	-61	-89	-147
	d	61	93	263	621
	e	33.3	14.5	87.9	84.6
	a			03.59	
inclin-	b		survey	-18	no
ometer	c		begins	-15	survey
tube	d			156	
	e			80.7	

a = bearing difference (MM.SS)
b = change in horizontal distance (mm)
c = change in vertical distance (mm)
d = magnitude of displacement (mm)
e = rates of displacement/month (mm)

FIG. 29: MIKONUI EARTHFLOW SURFACE DEFORMATION



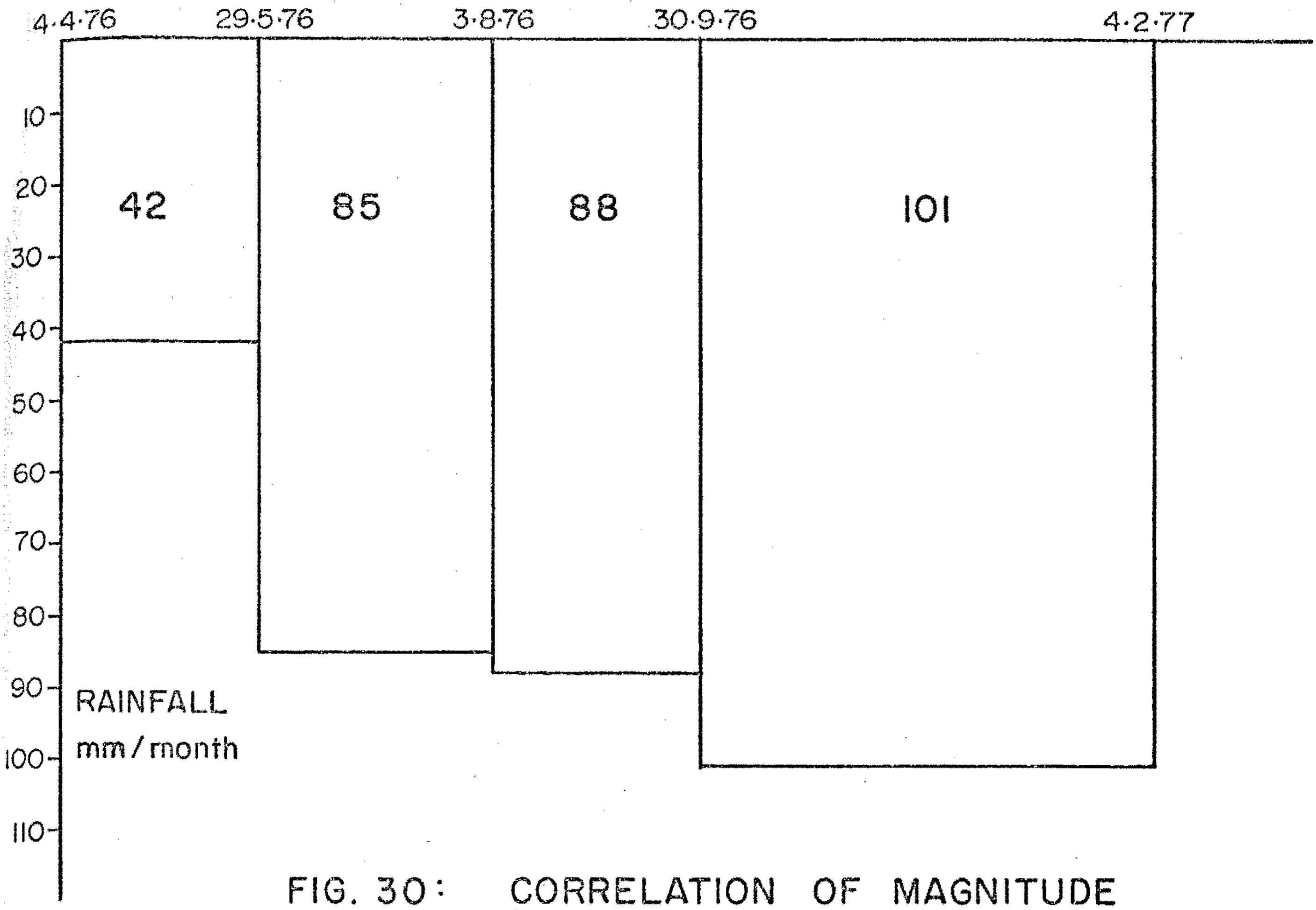
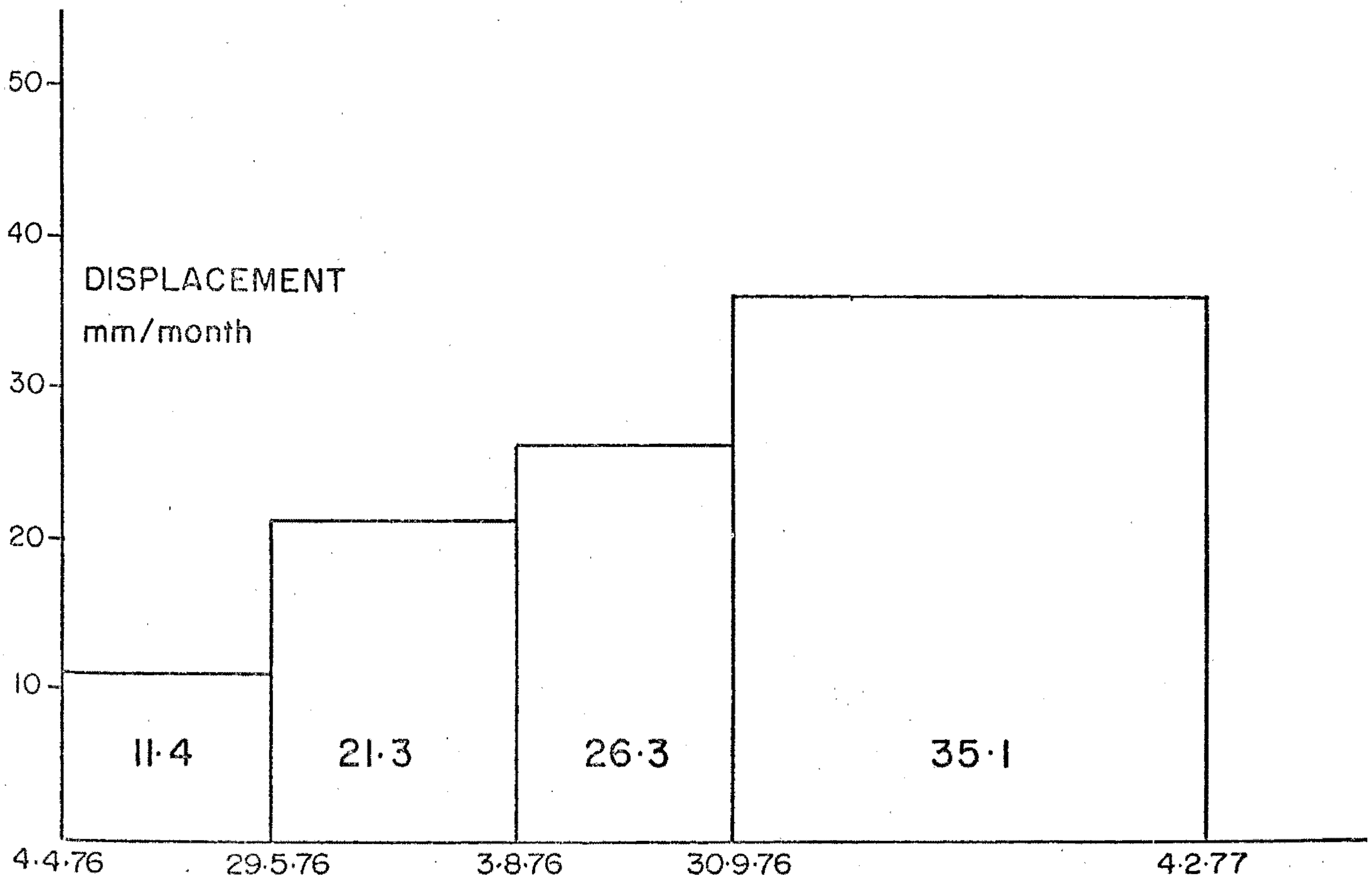


FIG. 30: CORRELATION OF MAGNITUDE OF MOVEMENT WITH RAINFALL, STATION NO 3, MIKONUI EARTHFLOW



toe are less than in upslope, narrower regions; and displacements adjacent to the margins are less than in areas closer to the centre of the earthflow.

(f) Rates of movement of markers per month are highly variable, ranging between o.5-215 mm/month, and as yet appear to bear no relationship to the amount of precipitation received in the general catchment. The only exception to this, on marker No. 3, rates of movement, fortuitously or otherwise, appear to correlate with rainfall (Fig. 30). A longer survey period, however, will be needed to effect any general correlation.

(g) Rates of deformation of the inclinometer access tubing (section 4.42) measured parallel to the uppermost station are:

27.7.76 - 17.8.76	:	7.9 mm/ month
17.8.76 - 31.8.76	:	12.9 mm/ month
31.8.76 - 15.9.76	:	115.0 mm/ month

These values, in general agreement with displacement of surface markers, are therefore likely to record a good indication of actual deformation rates within the earthflow at depth.

4.6 LABORATORY TESTING

Laboratory testing of soil samples obtained from exposures and drill hole cores were carried out by the writer and Central Laboratories and the Christchurch Soils Laboratory, Ministry of Works and Development.

The classification tests, moisture content, dry density, grain size and Atterberg limits, were performed according to the New Zealand Standards Specifications (1976). Lineal shrinkage, a form of swelling test, is a measure of the shrinkage of a circular cylindrical soil sample after repeated air drying until three sets of readings show no change in the sample length. The specimen is then oven dried for 12-24 hours, after which the final reading is taken. Tables 14 a & b summarise results of classification testing.

Clay mineralogy was determined by X-ray diffraction analyses undertaken at the University of Canterbury and the N.Z. Geological Survey, Lower Hutt. Four samples analysed by the writer, representative of each of the upper and lower bentonite members located at the crown of the earthflow (section 4.32), revealed very abundant Ca-rich montmorillonite in all samples and traces of illite in one of the soils. A similar result was obtained from two borehole samples of bentonite underlying colluvium in the earthflow analysed by the N.Z. Geological Survey. Their analyses showed at least 95% of the clay fraction amorphous to X-rays was a Ca-rich montmorillonite, with traces of illite and clay chlorite in one of the samples.

Shear box testing to determine residual shear strength was performed by Central Laboratories, MWD, on a sample of colluvium, located 8.2m below the surface in borehole B, and bentonite, located 31.2m below the surface in borehole B. The depth of the bentonite sample was less than 1m below the colluvium-bentonite interface. Both samples were consolidated in the shear box under normal stresses of 50, 100 and 150 k Pa. The colluvium and bentonite were subjected to at least six and ten reversals of shear, respectively, with 12mm displacement on each reversal, under each confining pressure. With increasing number of reversals,

TABLE 14a

SOIL PROPERTIES, BOREHOLE A, MIKONUI EARTHFLOW

Laboratory testing by M.W.D.

Material	Depth below grd. sf. (m)	Moisture Content (%)	Dry Density (T/M^3)	Grain Size			Atterberg Limits			Lineal Shrinkage (%)
				Clay	Silt	Sand	LL	PL	PI	
colluvium	5.33	32	1.36	38	50	12	60	37	23	11.3
"	6.70		soft							
"	6.85	27.2	1.69							
"	7.46	25.8	firm							
"	8.53	17.7	hard	14	41	45	25	29	6	
bentonite	12.95	28.7	hard							
"	14.47	36.3	1.51	30	44	26	80	40	40	17.2
"	16.00	21.8	1.75	31	33	36				20.1
"	17.52	21.4	1.82	25	51	24				18.0
"	19.05	22.2	dense compact							
"	20.72	23.2	dense compact							
"	21.94	18.3	dense compact	18	44	38				

TABLE 14b

SOIL PROPERTIES, BOREHOLES B AND C, MIKONUI EARTHFLOW

LABORATORY TESTING BY M.W.D.

Material	Depth below grd. sf. (m)	Moisture Content (%)	Dry Density (T/M^3)	Grain Size			Atterberg Limits			Lineal Shrinkage (%)
				Clay	Silt	Sand	LL	PL	PI	
Borehole B										
Colluvium	6.70	21.2	1.69							
"	7.16	22.2	1.67	22	43	35				
"	8.22	23.6	soft)							
"	8.38	19.3	stiff)	42	37	21				
"	8.53	30.0	1.53)							
Borehole C										
Colluvium	2.43	45.5	1.23							
"	3.65	28.2	1.58	33	48	19	42	24	18	10
"	3.96	2.90	firm							
"	5.48	24.0	1.47							
"	5.79			18	36	46	41	24	17	6
"	7.16	35.0	1.41	35	40	25	49	33	16	9
"	8.22	42.6	1.31	48	42	10	66	28	38	13.2
"	10.05	65.0	firm	69	29	2	56	32	24	12.6
"	11.12	71.0	0.74							

no reduction in the shear stress was experienced for either of the samples; thus the colluvium and bentonite were both assumed to be at their residual strength. The results of classification and residual shear box testing undertaken by Central Laboratories are given in Table 15.

TABLE 15

CLASSIFICATION AND SHEAR BOX TESTING FOR MIKONUI EARTHFLOW SAMPLES, PERFORMED BY CENTRAL LABORATORIES, M.W.D.

	colluvium	bentonite
initial water content (%)	41	40
final water content (%)	25	
specific gravity	2.61	2.78
liquid limit (%)	53	174
plastic limit (%)	27	48
plasticity index (%)	26	126
grain size: clay	26	48
silt	41	47
sand	33	5
cohesion (kPa)	0	22
angle of internal friction (^{o})	31	13

Conclusions to the laboratory testing programme include:

(a) Colluvium is distinguished from bentonitic sediments by a higher natural moisture content, lower dry density, lower lineal shrinkage and lower plasticity indices. These differences are attributable to the low permeability of bentonitic clays (to account for low natural moisture), and the high montmorillonite clay concentrations of bentonites (to account for higher plasticity indices and lineal shrinkage values).

(b) The colluvium matrix consists of silt sized grains with varying proportions of clay and sand. However, some layers are predominantly either clay or sand rich, suggesting the likelihood of layers of high and low permeability zones within the colluvium.

(c) Montmorillontic sediments underlying the earthflow are predominantly clays and fine to medium silts, with some sand sizes. The grain size distribution of these sediments contrast markedly with their bentonitic correlatives in the crown region of the earthflow; in the crown region, field observations indicate bentonites consist essentially of clay size grains.

(d) The residual angles of interal friction of the colluvium ($\phi_r' = 31^{\circ}$) and the bentonite ($\phi_r' = 13^{\circ}$) seem consistent with material whose predominant grain sizes are coarse silt and fine sand, and clay and fine to medium silt, respectively. The residual cohesion value of the bentonite ($C_r' = 22$k Pa) suggests the bentonite to be overconsolidated.

4.7 SLOPE STABILITY ANALYSIS

4.71 Introduction

The three-borehole drilling programme and monitoring of the inclinometer access tube enabled subsurface profiles of the lower (toe) region of the Mikonui earthflow to be drafted (see cross-sections, Site Plan). As a result of the investigations, an infinite slope stability analysis to compute the safety factor of the landslide was deemed most suitable, due to the planar shape of the slide plane. Assumptions that make the infinite slope analysis (Fig. 32) possible are:

(a) Sliding of the failed material occurs on a surface of planar shape.

(b) The ground surface and slip surface are parallel.

(c) A piezometric surface runs parallel with the ground surface or seepage occurs horizontally out of the slope.

(d) The lateral soil forces, Rn and Rnh, are equal but opposite.

The factor of safety of a failure in an infinite slope is expressed as:

$$F = \frac{C' + (pgH \cos^2 \emptyset P - U) \tan \emptyset'}{pgH \sin\emptyset P \cos\emptyset P}$$

where: C' = cohesion

$\emptyset'$ = angle of internal friction, both in terms of effective stress

$\emptyset P$ = angle of sliding

p = mass density

g = acceleration due to gravity

H = height of slide mass

and: $U = p_w gH$ for horizontal flow out of the slope

$U = p_w g Z_w \cos^2 \emptyset P$ for seepage parallel to the slope

where p_w = density of water

Z_w = height of water above the shear surface

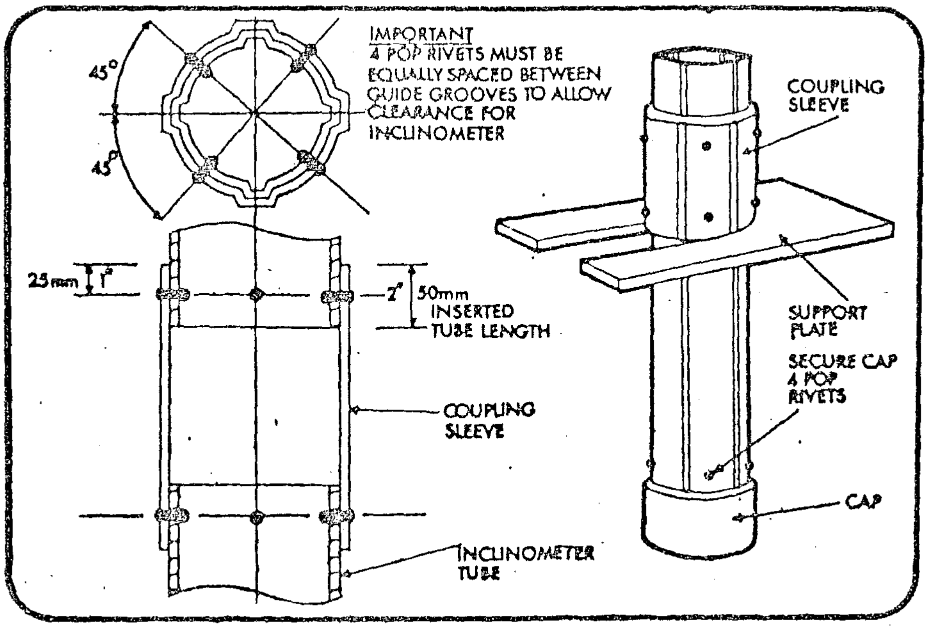
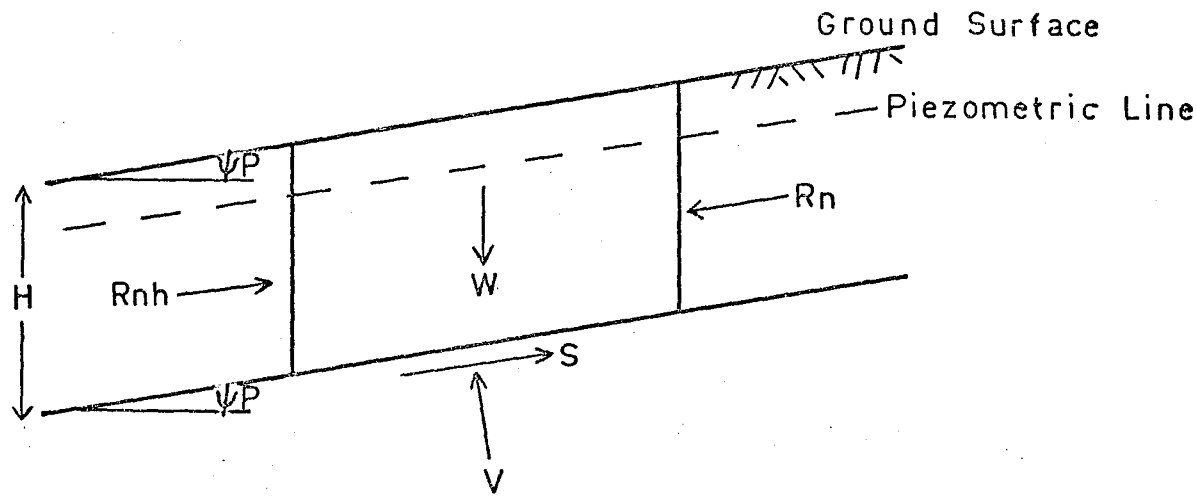


Fig. 31. Inclinometer access tubing.



where: S = shear strength of sliding plane
V = uplift force due to water pressure
ψP = slope angle, slide angle
W = weight of soil
H = slope height
Rn = symmetrical lateral forces
Rnh = symmetrical lateral forces

FIG. 32: INFINITE SLOPE ANALYSIS, MIKONUI EARTHFLOW (BASED ON HAEFELI, 1948)

More complex methods of analysis of earthflows in general are currently being studied. These include analyses using viscous flow models and analyses based on the theory of plasticity (Brunkl and Scheideggar, 1973). However, such complex techniques are outside the scope of this study.

4.72 Infinite Slope Analysis

Factors of safety by the infinite slope method were computed for the earthflow. The safety factors of the slope were calculated for three cross-sections through the lower (foot) region of the landslide obtained as a result of the three-borehole drilling programme. The piezometric pressures acting above the shear surface were calculated from average water table levels based on measurements over the time of this study. The piezometric surface was assumed to be parallel to the ground surface.

Through each cross-section, two sets of safety factor were computed, based on the differing densities and shear strength parameters of the colluvium and the bentonite.

Monitoring of surface survey stations and the inclinometer access tube show movement in the earthflow to be taking place. The safety factor of the slope is therefore known to have a value equal to, or less than, unity (F = 1.00 or less). However, drill cores did not reveal recognisable samples of the shear surface from which strength parameters could be determined and hence slope stability analyses could be computed. The safety factor of the slope has therefore been calculated using both colluvium and bentonite soil properties in order to determine the likely shear strength of the failure surface.

Factors of safety of the slope based on both colluvium and bentonite soil properties are given in Table 15. Parameters used in the analyses include:

colluvium: $p = 1.49\ T/m^3$

$\phi'_r = 31^\circ$

$C'_r = 0$

bentonite: $p = 1.69\ T/m^3$

$\phi'_r = 13^\circ$

$C'_r = 22$ kPa

Borehole A: $H = 15m$

$Z_w = 9.95m$ head

Borehole B: $H = 30.4m$

$Z_w = 15.4m$ head

Borehole C: $H = 20m$

$Z_w = 16.81m$ head

slide angle: $\phi P = 10^\circ$

TABLE 15

FACTORS OF SAFETY BASED ON INFINITE SLOPE STABILITY ANALYSES OF THE MIKONUI EARTHFLOW

cross-section location	safety factor (based on properties of colluvium)	safety factor (based on properties of bentonite)
borehole A	1.89	1.11
borehole B	2.25	1.17
borehole C	1.49	1.05

Table 15 clearly indicates the sliding surface to have shear strength parameters approximating those of the bentonitic material beneath the colluvium-bentonite interface. This result is possibly to be expected, as residual shear box testing has shown that a shear surface comprising bentonitic material offers less resistance to sliding than one comprising colluvial material. However, the safety factors of the slope based on bentonite shear strength parameters are still above a value of unity (F = 1.00); either bentonite residual shear strength parameters are slightly greater than those mobilized at the actual shear surface, or errors within the infinite slope analysis (for example, an incorrect height of water above the failure surface) are in operation.

4.73 Changing Groundwater Conditions

The quantitative effect of groundwater lowering through relief drainage on the stability of the toe region of the earthflow has been calculated. A reduction of the piezometric head above the sliding surface in order to increase the safety factors of the slope by 30% and 40% has been considered. Such increases in safety factor are of the right order of magnitude if long-term, permanent stabilization of the landslide is to occur.

Stability analyses by the infinite slope method were performed through the three cross-sections coinciding with boreholes A, B and C. Shear strength parameters of the sliding surface at each cross-section were taken from those obtained for bentonitic sediments located beneath the colluvium-bentonite interface. Results of groundwater lowering are given in Table 16.

TABLE 16

EFFECT OF GROUNDWATER LOWERING ON SAFETY FACTOR

FIGURES IN HEIGHT OF GROUNDWATER ABOVE SHEAR SURFACE

	borehole A	borehole B	borehole C
F	1.11	1.17	1.05
Present ground water condition	9.95m	15.4m	16.81m
F x 30%	1.44	1.52	1.37
Groundwater condition	7.49m	1.75m	8.45m
F x 40%	1.55	1.56 (33%)	1.47
Groundwater condition	5.34m	0m	5.87m
Thickness of slide mass	15.0m	30.4m	20.0m

Analyses through cross-sections based on boreholes A and C indicate that significant safety factor increases will only be achieved by a reduction of the groundwater above the shear surface to a level at least half that of the present groundwater condition. Analyses through the cross-section based on borehole B indicate that significant safety factor increases will only be achieved by lowering the groundwater to a level approximately coincident with the shear surface.

Reduced groundwater levels required to bring about significant increases in the safety factor of the slope would only be obtainable by the installation of subsurface drainage. However, as the groundwater level reductions needed to significantly increase slope stability are of a considerable magnitude, the implementation of suitable sub-surface drainage may be impractical from an engineering viewpoint. The reasons for this are discussed in section 5.24.3 and 5.33.

4.8 CONCLUSIONS

Engineering geological investigations have delineated an extensive area of earthflow-type slope movement in a catchment adjacent to the Mikonui Point area. The magnitude of the area involved in the slope activity was not appreciated before these investigations.

Material within the landslide comprises a colluvium, derived by erosion of insitu siltstones, sandstones and bentonites of the Okarahia Sandstone above the lateral and upper boundaries of the earthflow. A discreet zone of shear nearly everywhere defines the boundary at the ground surface of colluvium with in-place country rock. In lower regions of the landslide, colluvium attains a thickness of over 30m.

Sliding of colluvium in the lower part of the earthflow takes place above a layer of firm, dense bentonite. Bentonitic soils, consisting essentially of Ca-rich montmorillonite, are not involved in the earthflow movement. Bentonitic sediments in turn overlie hard, indurated sandstone of the Torlesse Supergroup.

A geological structure unfavourable to bedrock stability and the low shear strength of montmorillonitic clays are the actual causes leading to the formation of the colluvium.

Landsliding in slopes above the head and upper lateral boundaries of the earthflow result in slope debris accumulating in the upper half of the landslide. The weight of this accumulated debris provides the gravitational driving forces triggering movements in the colluvium lower in the slope.

Increases in hydrostatic pressure, especially within higher permeability layers in the colluvium, possibly control rates of displacement of the landslide, though no evidence during the time of this study could be found for this.

During the 10 month period of this investigation, surface survey markers in the lower landslide region recorded down-slope displacements between 13.2cm and 91.1cm. To date, movements have been slow, but continuous (creeping). Borehole deformation studies indicate the colluvium, from the ground surface to a depth just above the failure surface, moves at a rate comparable with ground survey stations. A correlation of rates of displacement with precipitation was not established.

Laboratory soil testing has shown colluvium to be distinguishable from underlying bentonitic soils on the basis of natural moisture, dry density, Atterberg limits, lineal shrinkage and shear strength.

Infinite slope stability analyses of the slope indicate the shear surface to have a shear strength comparable with bentonitic soils beneath the colluvium-bentonite interface. The computed safety factors of the slope range between 1.05 and 1.17, though monitoring of surface survey stations indicate the landslide to have a safety factor at least equal to a value of unity, or less. Considerable reductions in the groundwater level are required to effect significant increases in the stability of the lower (foot) areas of the earthflow.

SECTION 5: SLOPE STABILITY REMEDIAL MEASURES WITH RECOMMENDATIONS FOR EACH SITE

5.1 INTRODUCTION

Landslide correction measures, appropriate to one or more of the areas of this study, are listed in the first half of this section. Remedial measures unsuitable for any of the sites are also noted. Documented case histories of successful landslide stabilizations are used to illustrate several of the correction techniques.

The second part of this section outlines the preferred stabilization measures for each of the study areas. A slope reduction scheme is presently being considered for the control of batter instability at the Hawkswood Cut; recommended designs for the scheme were presented in section 3.7. This section outlines relief drainage to accompany slope reduction at the Hawkswood Cut.

At present, relief measures at the Ethelton Slip and Mikonui earthflow are not being contemplated. Recommendations for these areas are given with the knowledge that total stabilization of the landslides may be unacceptable on a cost-benefit basis. However, implementation of some of the lesser expensive recommendations (surface drainage) is likely to effect at least a partial control of the slope movements. Given the Main North Line has a finite life span, partial landslide control, coupled with an efficient maintenance programme, may be considered by N.Z. Railways to be economically the most acceptable relief scheme at the sites of the Ethelton and Mikonui landslides.

5.2 REMEDIAL MEASURES

5.21 Vegetation and Slope Stability

Vegetation is believed to affect slope stability in the following ways:

(a) Increase soil shear strength by mechanical reinforcement from plant roots. In deep seated slides, slope stability increases only if the root system penetrates the failure surface.

(b) Rainfall interception to minimise near-surface erosion as a result of slope runoff; especially beneficial under canopy-type vegetation.

(c) Depletion of the natural soil moisture as a result of transpiration.

(d) The effect of surcharge (weight of trees) on slope stability is generally beneficial, particularly if conditions of saturation in the slope are approached (Gray, 1974).

(e) Wind throwing or root wedging, the only disadvantageous effect of vegetation on slope stability, may be severe in areas of thin soil cover.

5.22 Elimination Measures

5.22.1 Relocation. Of all remedial measures, relocation of a highway or railway away from a landslide to a stable foundation will provide the most permanent solution to a slope stability problem. Alternative routes should be investigated for their own stability problems before relocation is proceeded with.

Although feasible from an engineering viewpoint, relocation of the Main North Line at any of the study areas would not be viable economically.

5.22.2 Bridging. Bridging a landslide will result in a solution to a slope stability problem as permanent as relocation. However, like all bridging constructions, costs will be high.

Bridging is usually undertaken over small landslides where bridge lengths up to 100m are involved. Piers may be placed in the moving mass if the overburden is not great, and the lateral soil driving forces against the piers are not excessive. The stability of the bridge abutments must be investigated before construction.

At the Hawkswood Cut, elevation of the railway above present base level on a series of piers was considered as a permanent solution to the batter stability problems. The design allowed slope debris to come to rest beneath the elevated track. The proposal has since been abandoned.

5.23 Slope Treatment

Excavation methods increase slope stability by reducing the driving forces that cause movement (Fig. 33). Stress reduction techniques find most application in the prevention rather than the correction of earth movements.

Where the correction of a landslide is sought, excavation methods find most use in the control of landslides having curved slip surfaces, due to higher tangential driving forces acting on the upper portion of the slope. Removal of the head (Fig. 33a), slope flattening (Fig. 33b), or slope benching (Fig. 33c), greatly reduce these driving forces causing movement. If the landslide is small, complete removal could be contemplated (Fig. 33d).

Loading the toe of landslides having curved slip surfaces may be considered as an alternative to excavation methods. Loading the toe increases the shearing resistance of the landslide, rather than decreasing the driving stress.

For block glides (landslides with planar slide surfaces), the removal of the head, or the addition of material to the toe, are both unsuitable. Any portion of a block glide having a weight differential with the rest of the landslide causes movement to take place. Uniform removal of the slope (Fig. 33e), or slope flattening, will usually be the only excavation methods undertaken on block glides.

Excavation methods are not recommended for landslides classed as earthflows unless combined with drainage. Movement in earthflows generally occurs above a surface of planar shape, thus uniform slope removal or slope flattening will be the only excavation methods considered.

Excavation methods usually only effect a permanent solution to a particular slope stability problem when combined with drainage. Also, for the excavation of very large landslides, costs will be prohibitive. For the uniform removal of earthflows, 25-50% of the moving mass should be removed. If construction of a railway or highway across the toe of a landslide having a curved slip surface is inevitable, one to two times as much material should be removed from the head as excavated from the toe. The stability of slopes above the excavation must be also considered for control by slope treatment techniques.

Excavation measures are unlikely to be considered as

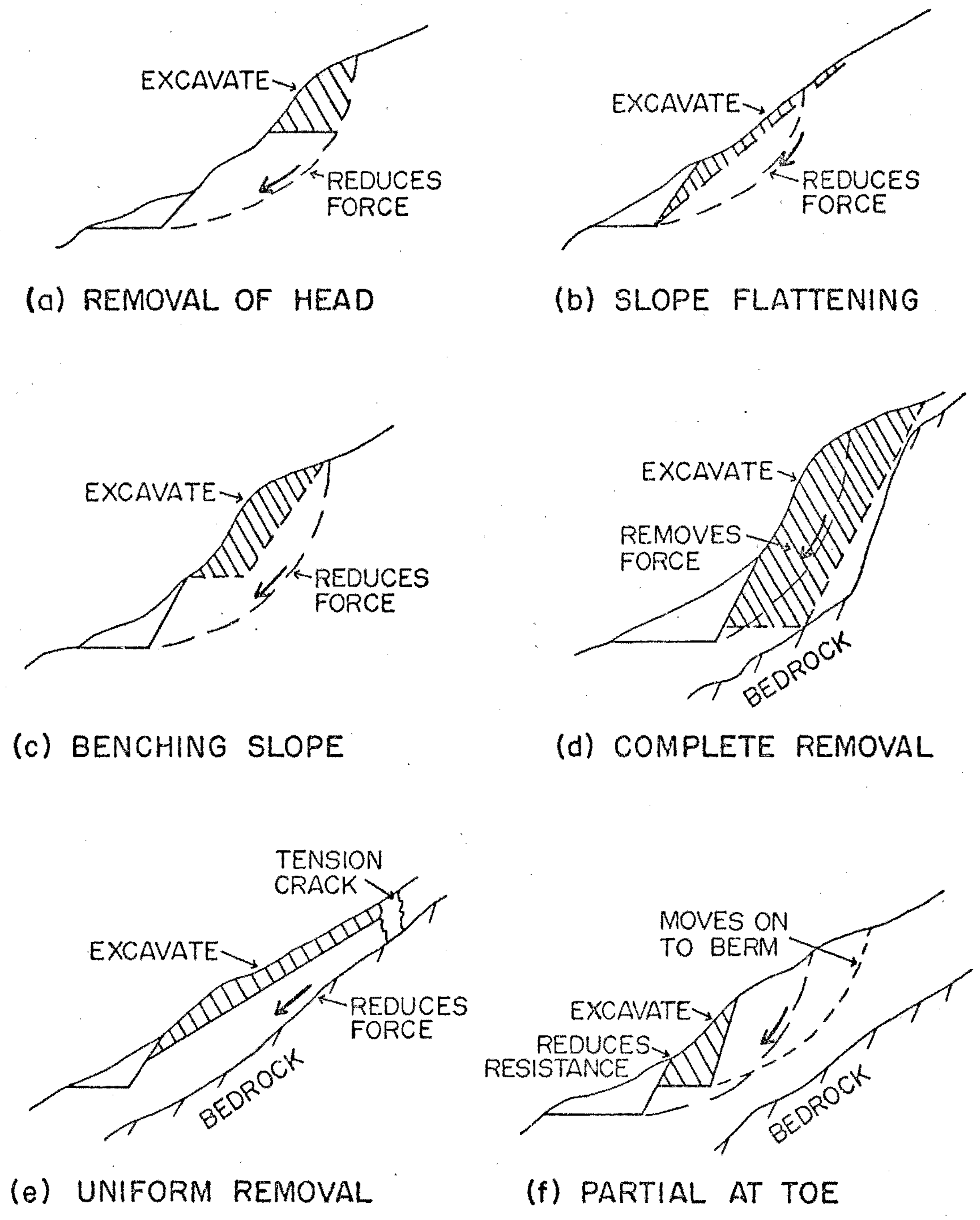


FIG. 33: EXCAVATION TECHNIQUES (FROM BAKER, 1960)

a control of the Mikonui earthflow. The lower (foot) region of the landslide is known from subsurface investigations to have a slide surface of planar shape; in middle and upper areas, the surface expression of the landslide suggests that while the slide surface may have a slight curved component, the shape will be predominantly planar. Loading the toe, or complete removal of the head, will therefore cause a weight differential with the rest of the landslide leading to a decrease rather than an increase in slope stability. Uniform slope removal or slope flattening are likely to be the only excavation methods considered, due to the nature of the failure. However, removal of 25-50% of the slope would be economically unacceptable, with the additional likelihood that landsliding in the slopes above the excavation is a possibility.

The Golden Slide, near Denver, Colorado, was only partially stabilized after the head of the landslide was unloaded and the excavated material placed at the toe (Noble, 1973). The safety factor improved by only 1% following these measures. Complete control was only effected after implementation of subsurface drainage.

5.24 Drainage

5.24.1 Introduction. The removal of water within earth movements by drainage is unquestionably the most advantageous measure for stabilization of landslides. Drinage may be preferred over all other measures because it is relatively inexpensive, and because results are usually expeditious.

Baker (1960) summarises the detrimental effects of water in earth movements. These are:

(a) Reduced shearing resistance through increased hydrostatic or pore pressure.

(b) Increased shearing stress by the addition of the weight of water to the sliding mass.

(c) Increased shearing stress due to the addition of seepage forces.

(d) Geochemical and physical changes in the landslide material (weathering effects).

Increases in pore water pressure in a slide mass appear to be universally accepted as the principal contributing factor causing slope instability. The effect of drainage on excess pore pressures, and hence slope stability, is given in the following analysis.

The resistance to shearing (S) of a soil mass on a potential slide surface is given in terms of the Mohr-Coulomb equation (section 3.51):

$$S = C' + \sigma' \tan \phi' \qquad \text{- EQ. 1}$$

where σ'= normal stress

C'= cohesion

ϕ'= angle of shearing, all three in terms of effective stresses.

The effective normal stress (σ') is related to the total normal stress (σ) and soil pore pressure (U) by:

$$\sigma' = \sigma - U \qquad \text{- EQ.2}$$

Resolving the two equations leads to the conclusion that a reduction in soil pore pressure (U) increases the resistance to sliding of the soil on the potential slide surface. Thus, drainage increases slope stability by promoting a rise in soil shear resistance.

The importance of drainage cannot be over emphasised. Nearly without exception, in earth movements where the construction of engineering structures is not the actual cause of failure, excess hydrostatic pressures are commonly the ambient conditions regulating slope movements.

Drainage may be installed on all earth movement-types, and is strongly encouraged for landslides classed as earthflows. Drainage is commonly used in combination with alternative correction measures. Drainage consists of ditches constructed on the landslide surface and its environs, and subsurface drains installed within the failed mass.

5.24.2 Surface Drains. Surface drains consist of ditches which permit the rapid runoff of surface water away from a slope. Surface drains are commonly constructed outside a landslide surrounding the upper and lateral boundaries. Here their function is to intercept runoff entering the unstable slope. Cutoff drains may have their downslope edge lined to prevent water infiltration into the landslide (Fig. 34a).

Ditches constructed on the landslide surface are aligned to the direction of movement, and are designed to remove precipitation falling in the landslide catchment.

All surface drains must be constructed to the following requirements:

(a) Drains must be levelled when installed to prevent sections of reversed flow.

(b) Drains on slopes greater than 10-15° may be left unlined provided removal of water is rapid, otherwise

(c) Drains must be lined with an impermeable seal (plastic membrance, concrete).

(d) After installing, drains must be maintained.

A cut-off trench constructed above the crest of the west batter, Hawkswood Cut (section 3.1), exemplifies the detrimental effects of improperly constructed surface drains. The trench, unlevelled, unlined, and unmaintained, is ponded throughout much of the year and is therefore contributing to, rather than reducing, slope instability in the cutting.

In addition to surface drains, sealing of surface cracks, drainage of ponds and swampy ground, and regrading of hummocky topography should be undertaken to encourage rapid runoff of surface water.

5.24.3 Subsurface Drainage. Subsurface drainage is installed either to intercept subsurface water flows entering an earth movement, or to lower the piezometric head above a slide surface.

Subsurface drainage will only be successful if the drains reach the water-bearing horizons; the soil mass is relatively permeable; and the drain is located in unyielding material so that dislocation of the structure is prevented.

Subsurface drainage will find most application in the control of large earth movements where installation costs are typically ¼ to ½ those of alternative methods. For relatively small landslides, complete removal of the slope is commonly more economical than subsurface drainage.

(a) Counterfort drains. These consist of excavated trenches, 1m wide, aligned in the direction of movement (Fig. 34a). Trenches should be deep enough to intercept the slide plane (impractical for depths greater than

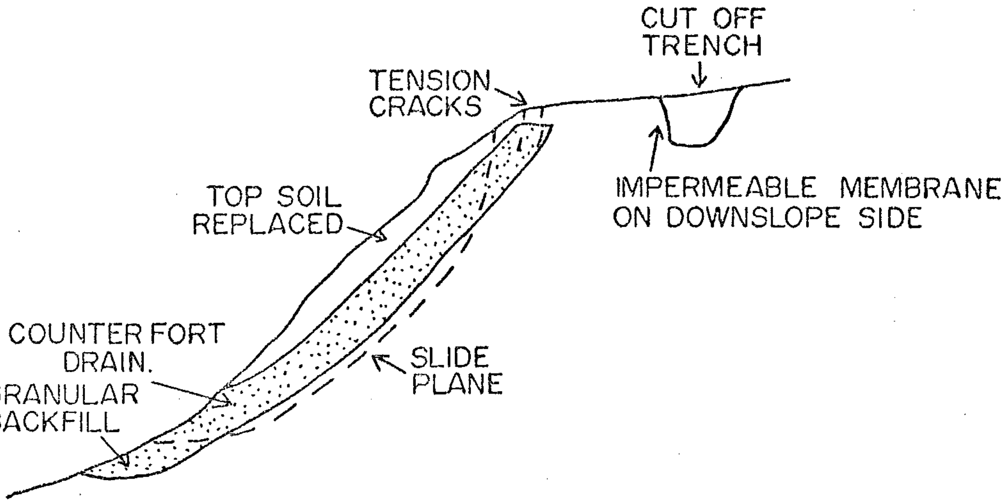


FIG. a. INTERCEPTOR TRENCHES

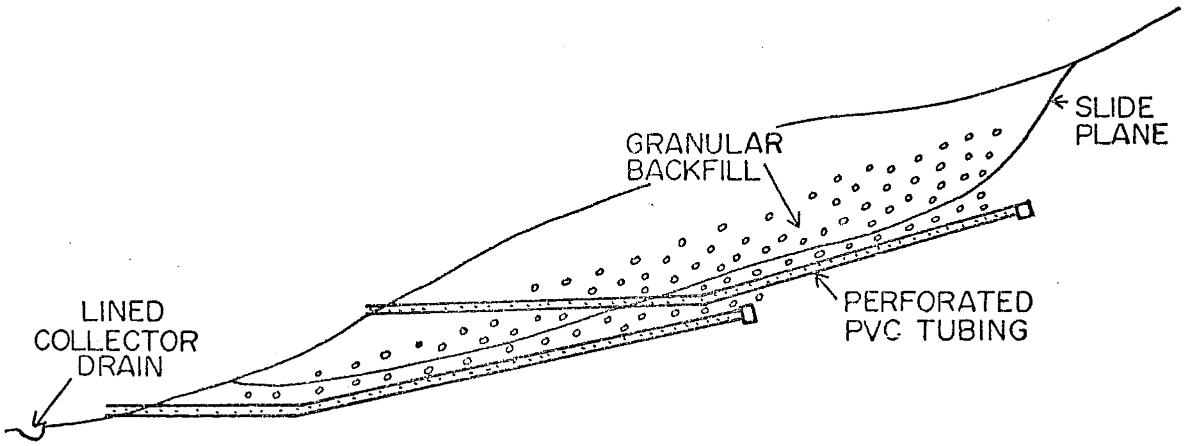


FIG. b. DRAINAGE TRENCHES

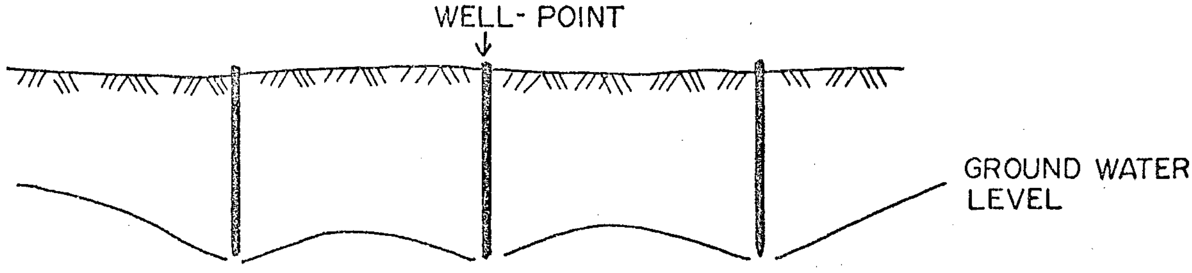


FIG. c. VERTICAL WELL - POINTING

FIG. 34: SUBSURFACE DRAINAGE METHODS

approximately 6m), and extend several metres above the upper landslide boundary to drain tension cracks. Trenches are backfilled with free-draining, well graded granular fill. Top soil is replaced above the fill to permit grassing.

On large movements, a number of trenches are spaced at intervals across the slope.

(b) Drainage trenches. Drainage trenches are a modification of counterfort drains, consisting of a trench in which a perforated PVC pipe (5.0cm diam.) is laid on the bottom. The trench is then backfilled with free-draining fill. The pipe should be located beneath the slide plane to prevent dislocation of the drain. Pipes should permit gravity flow of water.

(c) Vertical wellpoints. This method consists of installing at close centres a number of small wells (4cm diam., 1m long), secured to the ends of riser pipes of appropriate lengths, into the soil (Fig. 34c). Riser pipes are connected via a common suction header pipe to a wellpoint pump.

Vertical wellpointing is usually undertaken as a dewatering measure associated with construction sites. The method is not recommended for permanent landslide drainage due to the high costs involved in continuous pumping. The technique may be used for short term control, for example during the removal of the toe of a landslide until completion of a buttress.

(d) Horizontal drains. Horizontal drains consist of a perforated PVC tube (5cm diam.) placed in a borehole, slightly inclined from the horizontal to permit free drainage. Boreholes must be sited to locate all pockets of perched groundwater.

Horizontal drains are installed either in rows, typically at 3m centres, or as a number of drains radiating from a central point (fanned).

(e) Vertical drains. These comprise vertical, auger holes of diameter 0.5-1m, bored to depths up to approximately 30m. Holes are either lined with a perforated steel casing, or backfilled with sand or other filter medium (Fig. 34e).

Vertical drains are commonly combined with horizontal drains or drainage galleries, these permitting gravity flowage of water from the base of the vertical drain. Where free drainage is not possible, submersible pumps are needed to remove the water.

Vertical drains are used frequently as a drainage curtain above the upper boundary of a landslide. A series of drains interconnected at their base, aligned across the slope, prevent infiltration of water into the landslide.

(f) Drainage galleries. Tunnelling is used primarily to intercept deep-seated moisture flows. The method is the most expensive subsurface drainage measure, and is used only in the protection of expensive structures. Drainage drives are often combined with vertical sand drains and horizontal drains.

With the possible exceptions of counterfort drains and steel-lined vertical shafts, subsurface drainage appears impractical for control of the Mikonui earthflow. The magnitude of the landslide would prohibit successful draining by horizontal drains; uncased, vertical sand drains would experience disruption and dislocation due to shear displacements known to be taking place within the earthflow; drainage

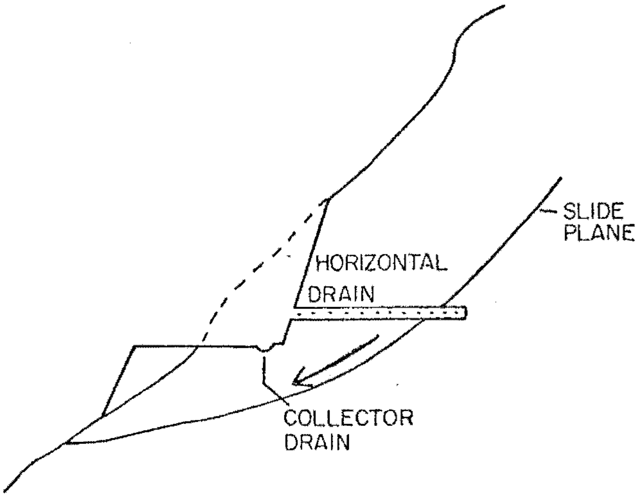


FIG. d : HORIZONTAL DRAIN

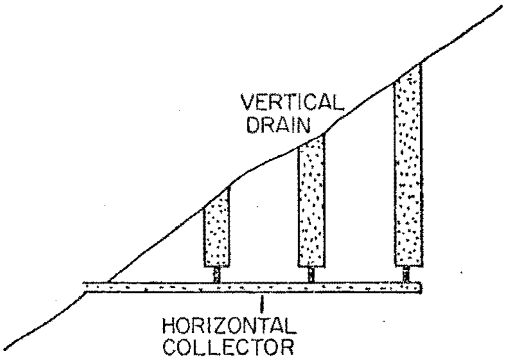


FIG. e : VERTICAL SAND DRAINS

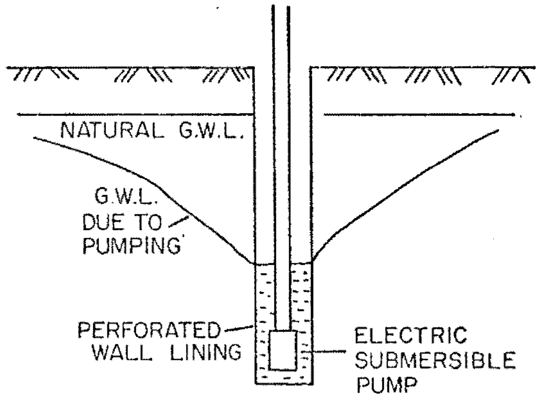


FIG. f : DEEP FILTER WELL

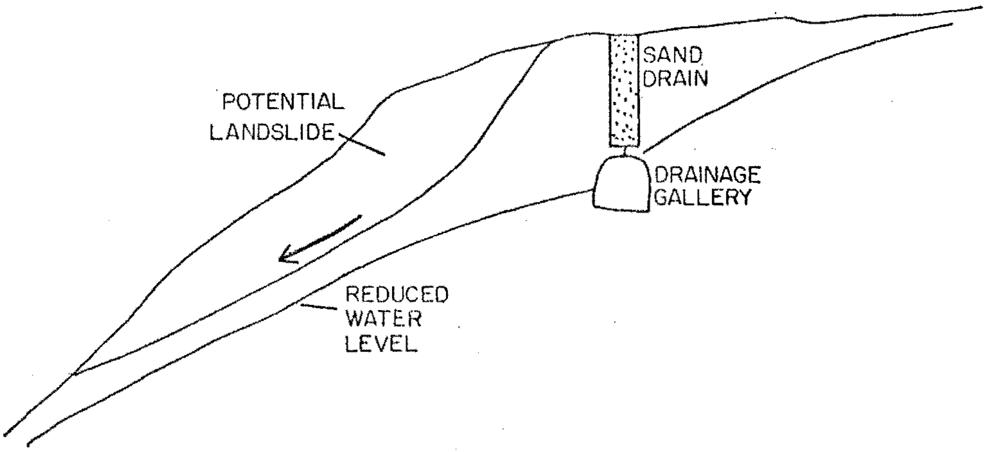


FIG. g: DRAINAGE GALLERY AND VERTICAL SAND DRAIN

FIG. 34: SUBSURFACE DRAINAGE METHODS

trenches would similarly be unsuited, as over most of the site a firm, unyielding foundation for the base of the drain would be at a depth greater than that reached during the installation of drainage trenches.

Numerous case histories appear throughout the engineering literature documenting the successful control of earth movements through drainage. D'Appolonia et al (1967) report on the construction of a surface and subsurface relief drainage scheme in the prevention of a landslide in a colluvial slope in Weirton, West Virginia. A large excavation up to 20m high and 800m long was planned to be constructed at the toe of the slope; stabilization measures to prevent re-initiating movement of an ancient landslide were proposed.

The drainage scheme comprised a surface cutoff trench near the crest of the slope to intercept groundwater entering the potential failure zone. The cutoff trench was underlain by a drainage gallery, the two connected by open drain holes. A second drainage gallery was located half way down the slope. At the toe of the slope, vertical sand drains were installed to relieve pore pressures immediately behind the excavation.

5.25 Restraining Structures

After drainage, restraining stuctures are the most frequently used landslide stabilization measure. One North American agency (Oregon State Highway Division) favours the counterbalance (buttress) method as the most positive measure to arrest earth movements, especially smaller-type displacements (J. H. Versteeg, pers. com.).

Restraining structures increase slope stability by increasing the shearing resistance of landslides through impeding and obstruction of earth movements. For this reason, most restraining structures will be located at the toe of a landslide. A lack of understanding by many engineers of the magnitude of the driving forces mobilised in mass earth movements, borne by restraining structures, has led to many dismal failures with this stabilization method.

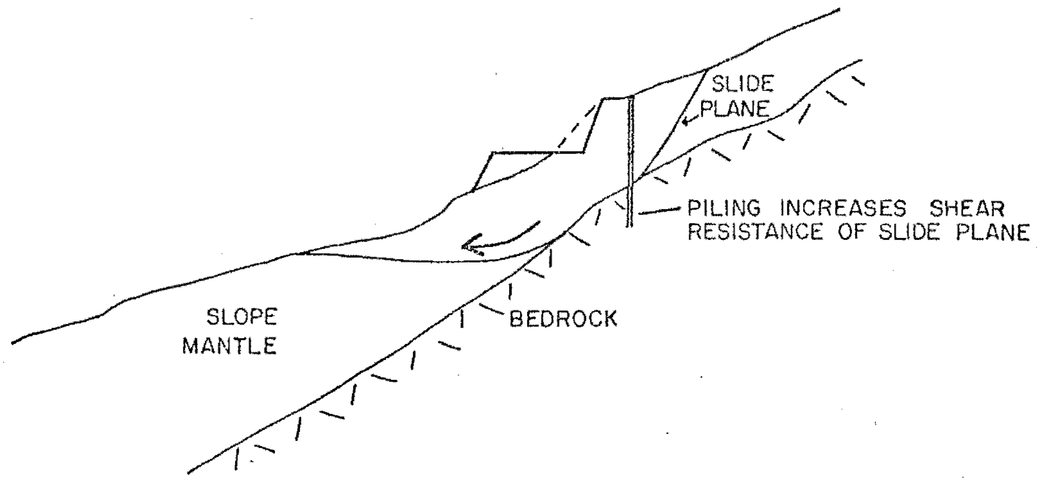
The three principal restraining methods are (Fig. 35):

(a) Buttress. Almost without exception used at the toe of landslides for correction purposes. Material removed from the toe is replaced with earth or rock fill. Fill material should have an angle of internal friction greater than the slide surface. The base of the buttress must be located in bedrock underlying the landslide, or at least 4-5m beneath the slide surface. Buttrresses should have a volume ¼ to ½ of the moving mass, thus they are particularly suited to smaller type landslides.

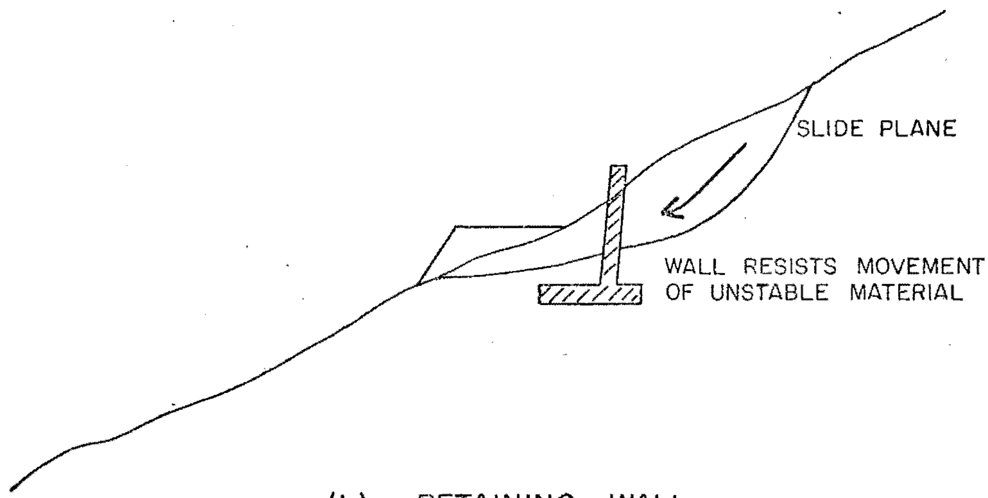
Buttresses have had creditable incidences of success in the correction of block glides or landslides having curved failure surfaces. They are not recommended for control of landslides classed as flows, due mainly to the plastic, remoulded nature of the failure.

(b) Crib and retaining walls. These measures find most use for the prevention of failures in excavated slopes. The base of the structure must be sited on firm, unyielding material below any potential failure surface. Retaining walls are limited to the control or prevention of landslides of small magnitude.

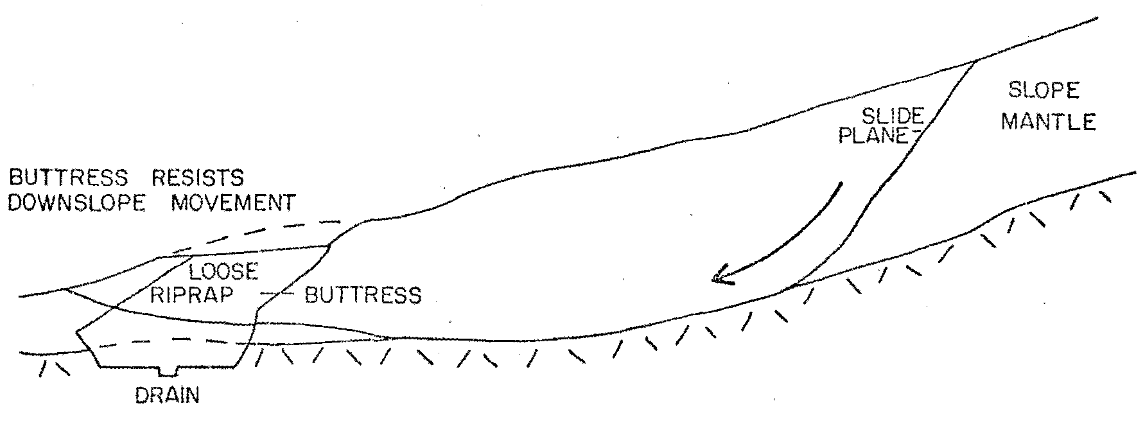
(c) Piling. Piles driven through a slide to increase



(a) PILING



(b) RETAINING WALL



(c) BEDROCK BUTTRESS

FIG. 35: RETAINING STRUCTURES

the shearing resistance of a soil mass have been perhaps the most widely mis-used landslide correction measure. Few are the cases where piling has resulted in the successful stabilization of massive earth movements. Piles are likely to be ineffective through, plastic flowage in earthflows around piles, overturning of the piles, by shear failure of piles, or by displacement of the shear surface to a depth below the pile tips.

Piles find most application in the prevention of small slides before displacements of any magnitude have occurred. Should a slide surface have reached its residual shear strength, or the soil mass be in a remoulded state, piles are likely to be ineffective.

Restraining structures would not be suitable as correction measures in the control of the Mikonui earthflow, due to the known magnitude and nature of the slope failure. Retaining walls and piling would similarly be unsuitable for use at the Ethelton Slip due to the magnitude of the landslide.

Squier and Versteeg (1971) report the successful control of the OMSI - Zoo landslide, located two kilometres west of the city centre of Portland, Oregon, by the construction of a rock buttress. Massive earth movements were triggered in 1957-58 following regrading of the slope for a museum and zoo facilities, and widening of a highway to service the facilities at the toe of the slope. Following the initial investigations (drilling, slope deformation studies, laboratory testing), a relief drainage scheme, comprising a series of .76m diam., 24-30m deep, vertical sand drains at 2.4m centres, was installed. A reduction in the landslide's activity was obtained initially, though this deteriorated proportionally with a reduction in the effectiveness of the drainage curtain through fines plugging the filter material.

Full stabilization of the OMSI - Zoo landslide was not attained until construction of a rock buttress in 1970. The buttress was located at the toe of the slope and consisted of fairly clean quarry run basalt. The buttress extended down to bedrock underlying the slip surface, and was designed to increase the safety factor by 25%.

5.26 Unconventional Methods

5.26.1 The electro-osmotic effect. Electro-osmosis is used infrequently as a dewatering measure associated with engineering construction sites. In November, 1964 the technique was successfully employed in the reduction of excessive pore water pressures in the 25m high West Branch Dam embankment, thus enabling completion of the project (Fetzer, 1967).

Electro-osmosis is based on the well-known phenomenon of introducing a direct electrical current between both openings of a water-filled, U-shaped flask. Water in the flask flows towards the negative electrode (cathode). This is the basis for electro-osmotic dewatering plants (Fig. 36a).

Myslivec (1969) and Veder (1973) report a naturally occurring electro-osmotic effect associated with earth movements. Under certain soil conditions, an electric field can arise on contact of mobile and immobile layers of soil in a landslide. The potential difference as a result of this electric field causes movement of water in an upward direction and, consequently, an increase in the pore

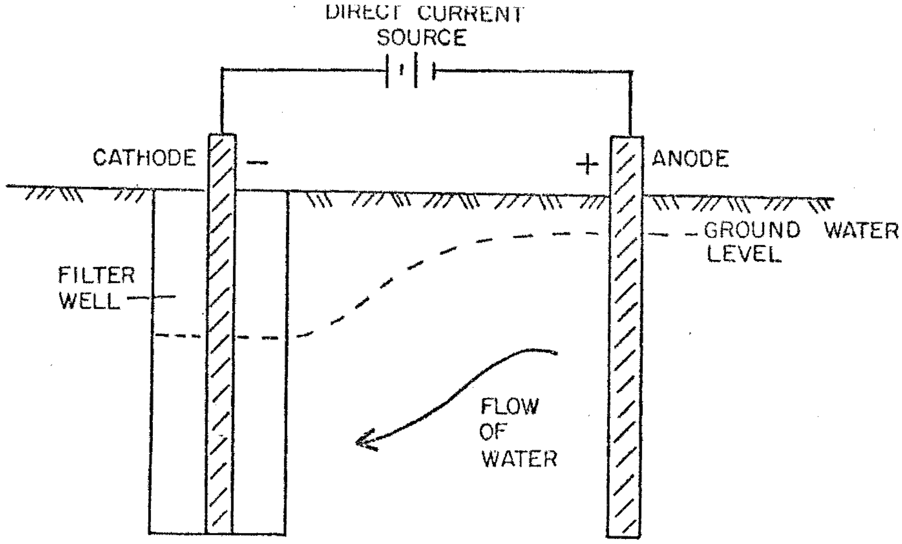


FIG. 36 a. FILTER WELL RUN ELECTRO-OSMOTICALLY BY EXTERNAL CURRENT (AFTER VEDER, 1973)

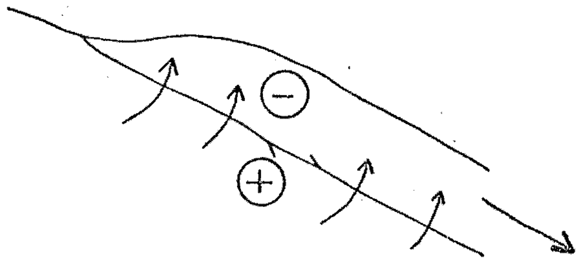


FIG. 36 b. BUILD UP OF POTENTIAL DIFFERENCES BETWEEN SLIP AND STABLE GROUND CAUSING UPWARD MIGRATION OF WATER (AFTER VEDER, 1973)

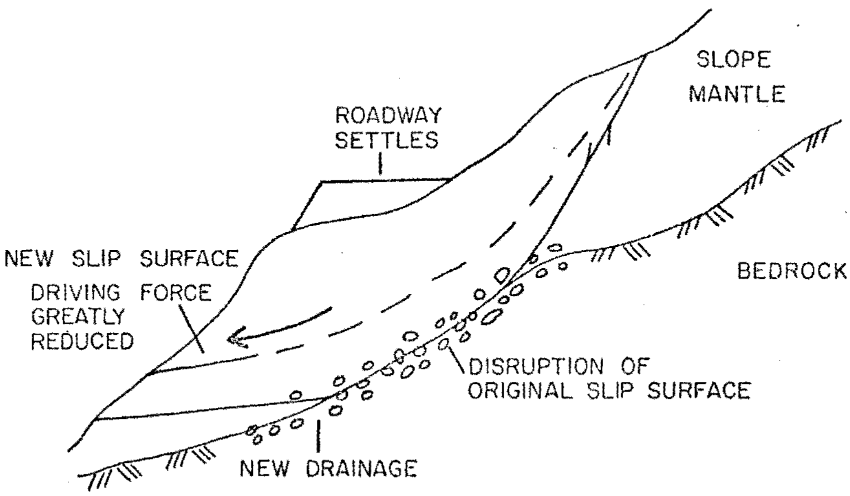


FIG. 36 : BLASTING TECHNIQUE (AFTER BAKER 1960)

pressure above the slide surface (Fig. 36b).

Both Myslivec and Veder report the stabilization of landslides through nullification of this potential difference. Short-circuiting is achieved by driving aluminium or iron rods through the slide and partially into the immobile layer beneath the failure surface. A rapid reduction in the pore pressure within the mobile layer is reported following short-circuiting.

The phenomenon of a potential difference between contact zones in landslides is unfamiliar to New Zealand engineering personnel. The technique would be considered unlikely as a control measure at any of the study areas without prior research into the reasons for its reported success. However, the technique should not be overlooked, as its principal advantage lies in the minimal cost associated with obtaining lengths of metal rods (lengths of railway track would be suitable).

The technique would be unsuitable for use at the Mikonui earthflow (typically the depth to the slide surface should not exceed 3-4m), and at the Hawkswood Cut (wrong type of slope failure).

5.26.2 Blasting. Partial or complete disruption of a failure surface by blasting remains the most contentious landslide correction measure. The technique has had results ranging from spectacular success to dismal failure.

Blasting is most applicable to landslides of relatively small lateral extent, where the slip surface is at a depth 4-15m below ground surface. The technique is not suitable for earthflows.

The following important factors should be considered before blasting is undertaken:

(a) Firm bedrock should underlie the slide surface for any real chance of success.

(b) The slip surface may be disrupted and relocated to a higher elevation. Relocation of the failure plane may increase stability by increasing the soil shear resistance and decreasing the shearing stress.

(c) Improved drainage at the depth of blasting will reduce pore water pressures. However, improved drainage will possibly be of a limited duration due to blocking of the soil mass by fines.

(d) Settlement of the ground surface must be expected, mostly within one year.

(e) Blasting is likely to be advantageous due to costs commonly 1/10 to 1/4 those of alternative methods. For this reason, a series of blasts over a number of years is possibly more economical than a single correction by an alternative technique.

5.3 RECOMMENDATIONS

5.31 Ethelton Slip

The limited site investigation to date precludes any but the most basic and inexpensive remedial measures being implemented. Expensive excavations and constructions cannot be considered until the shape of the failure surface and extent of movement are known, and slope stability analyses undertaken. For this end, further site investigations must be executed (see section 2.9).

The following recommendations are based on results of the investigations so far carried out:

(a) The area of swampy ground and seepage located at the head of the landslide should be drained by a surface ditch. To be practical, the ditch should be excavated with a fall towards the unnamed stream bisecting the landslide. The upper part of the ditch should be lined, as the slope gradient in this region is insufficient for free flowage along an unlined drain.

(b) Those areas in middle and upper regions of the landslide where slopes are reversed should be regraded to facilitate surface runoff and prevent ponding.

(c) Should any section of the unnamed stream be shown to lose water to the slide through infiltration, a flume or culvert over that section of the stream should be considered.

(d) All surface ditches adjacent to the county road and railway experience ponding after rainfall. The drains have an insufficient gradient to permit gravity flow of water, and should therefore be lined. Lengths of semi-circular concrete piping would appear suitable.

(e) All flumes associated with surface drains in the vicinity of the road and railway should be upgraded and subsequently maintained.

(f) The site is fully vegetated, though the plant cover is limited to scrubland species and grasses. Consideration could be given to either purchasing or leasing the land above the county road to enable the removal of grazing stock and replanting with a tree-type cover. Replanting would increase removal of moisture through transpiration, and the mechanical reinforcement of the soil by roots could help prevent surface cracking.

(g) Tension cracks and shear zones associated with the zone of activity (section 2.4) must be regraded, or infilled with an impermeable clay, to prevent infiltration of water into the slide mass. Periodic checks should be undertaken at this location, and over the whole landslide surface, so that additional areas of cracking may be noted and the displaced soil regraded.

(h) The site investigations to date indicate that deep seated movement is confined to a single zone of activity, of relatively small area, in the lower (foot) region of the landslide. Should re-surveys of surface markers in years subsequent to this study confirm that activity is confined to this zone, the installation of a relief subsurface drainage scheme could probably achieve slope stability. Due to the difficult nature of the terrain, horizontal drains would appear most applicable. Horizontal drains could be installed in rows at varying levels from both the road and railway. Current in place installation costs for horizontal drains are approximately $6.50 per metre.

(i) Should subsequent site investigations indicate that movement is not confined to a specific zone, but is occurring generally over much of the site, additional remedial measures will be necessary. Due to steep topography above the upper and lateral boundaries, a cutoff trench surrounding but back from the landslide to prevent water infiltration would not appear practical from an engineering viewpoint. A drainage scheme feasible from an engineering basis would be a series of counterfort drains or drainage trenches, spaced at intervals across the slope, and aligned in the direction of movement. The drains would require excavation to at least the depth of the sliding surface, and could only practically be installed from the head, downslope to the county road. Installation costs for counterfort drains compare favourably with those of horizontal drains; current in place costs for 1m wide, 5-6m deep, counterfort drains are approximately $6.90 per metre. On major slides they can only practically be installed in summer.

(j) Unconventional methods of stabilization should not be eliminated from consideration due to their low costs of installation. Site investigations to date suggest blasting and nullification of zones of potential difference both warrant further investigation.

Recommendations for remedial measures have so far concentrated on the elimination of water from the site, especially surface runoff. Inadequate drainage, leading to high hydrostatic pressures in the slide mass, is viewed by the writer as perhaps the most important ambient factor presently controlling movement. The implementation of well maintained surface drains, coupled with slope regrading where necessary, may possibly be found to be the only correction measures needed, without resorting to expensive subsurface drainage and excavation techniques.

5.32. Hawkswood Cut

The following recommendations accompany the proposed slope reduction scheme proposed as a control of batter instability at the cutting. A design for the proposed excavation was discussed in section 3.7. The recommendations are:

(a) A surface drainage and slope regrading scheme be implemented on the surfaces above both east and west slope crests.

(b) Slope regrading should consist of sloping the ground surface towards surface drainage ditches aligned in the direction of, but back from the cutting crests (Fig. 37). Such a design will allow precipitation falling between the slope crest and the ditch to flow towards the drain.

(c) All surface depressions beyond the areas of slope regrading must be levelled.

(d) Drainage ditches will run the length of the cutting at a distance back from both proposed east and west batter crests. The distance of each ditch from the slope crest is flexible, but should be sufficient to prevent any possibility of disruption through renewed batter instability.

Drainage ditches should be levelled to slope away from a relatively elevated point above the centre of the cutting towards both north and south approaches. At both approaches, flumes will be needed to carry water down the batters to the level of the track and away from the cutting.

Drainage ditches above the cutting should be lined, as their gradient will be insufficient to allow gravity flow towards both approaches.

(e) Surface drainage combined with slope regrading should be adequate to prevent all surface runoff, except precipitation settling directly on the batters, from entering the cutting. Hence, most of the earthflow-type batter instability, as a result of surface runoff leading to a reduction in the effective normal stress of the surficial slope material, should be eliminated.

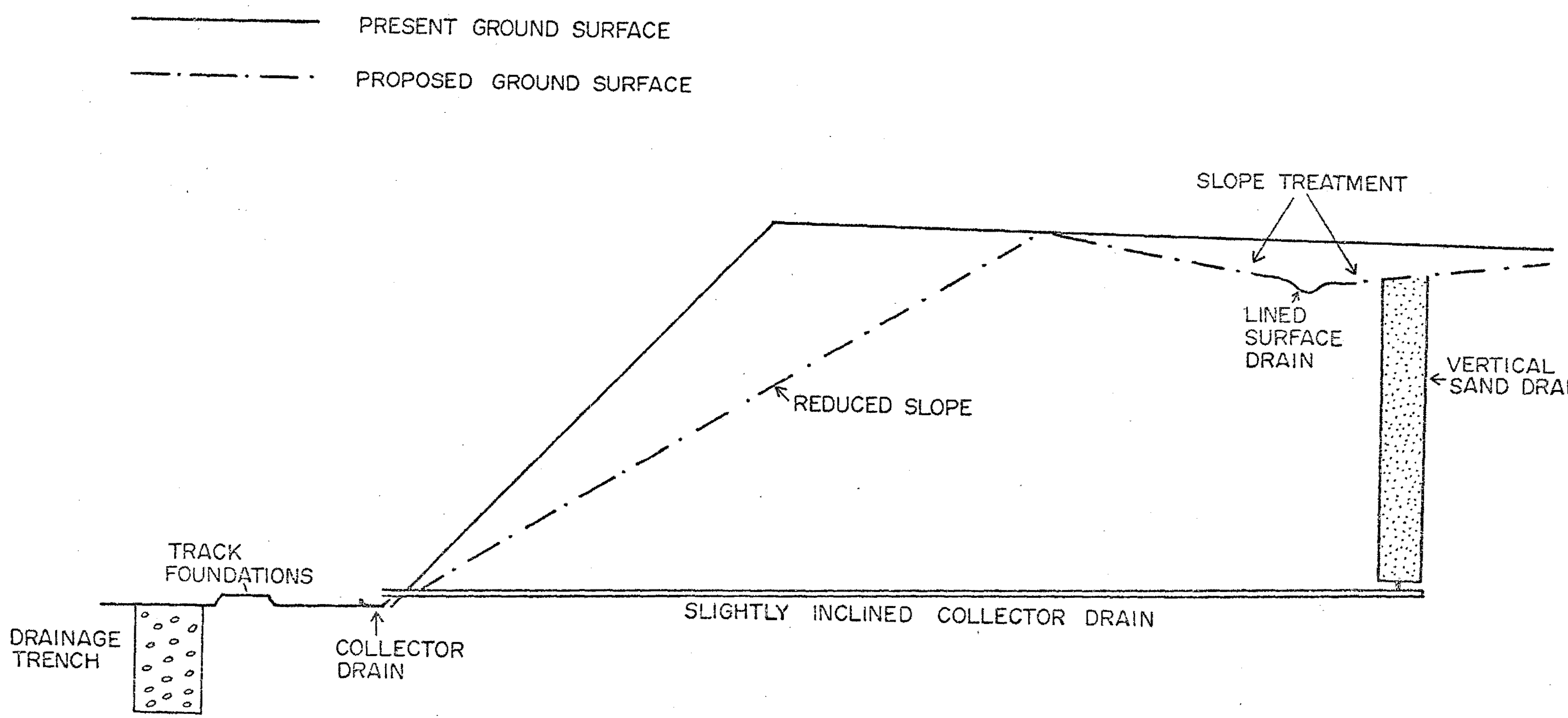


FIG. 37: SCHEMATIC DIAGRAM SHOWING RECOMMENDED REMEDIAL TREATMENT AT THE HAWKSWOOD CUT. Not to scale.

Should surface drainage and slope regrading not lead to a complete stability remedy, subsurface drainage should be considered as an additional protection for the new structure. The primary purpose of subsurface drainage will be in the reduction of pore water pressures in soils beneath the batter faces. Hence, subsurface drainage will be most applicable in the control of earthfall-type landsliding. Subsurface drainage, however, should only be considered as a last resort measure.

(f) Two subsurface drainage schemes could be considered. The first of these comprises an excavated trench (up to 6m deep) constructed the length of the cutting midway between the base of the slope and the railway (Fig. 37). The trench would be backfilled with a free-draining granular fill. The drain would fall towards the southern approach; an impermeable membrane at the base of the trench would ensure a relatively rapid and continuous flow. The effect of a backfilled trench on the stability of slip circle failures was discussed in section 3.62.

The preferred subsurface drainage scheme comprises a series of vertically bored sand drains aligned in the direction of the cutting at a distance back from the slope crests. The drain centres and diameter would be determined from vertical sand drain theory. Drains could be interconnected at their base by small diameter tubing, such that free flowage of water to selected collector sand drains occurred. Collector sand drains would be tapped by drilling a slightly inclined borehole from the batter face (Fig. 37). Lined surface drains would remove water from inside the cutting.

Vertical sand drains are preferred to drainage trenches as sand drains would pass through and hence intercept water within all layers in the vicinity of the batter faces. Sand drains would therefore act as an effective drainage curtain. Drainage trenches would act as a depressant on the phreatic level generally. However, as a number of phreatic levels are known to exist in soils adjacent to the cutting (section 3.44), the effect of drainage trenches on a reduction of hydrostatic pressures will differ markedly according to the soil permeability. In practice, therefore, trenches may not increase the safety factor as much as the values given in section 3.62.

5.33 MIKONUI EARTHFLOW

Landslides classed as earthflows are, generally, the most difficult of all earth movements to stabilize. This is due to the characteristically low resistance to shearing mobilized at the failure surface, and to the plastic, remoulded nature of the failed soil mass. Drainage is usually recommended for all earthflows.

Any stabilization scheme for the control of the Mikonui earthflow must be based on either decreasing the driving forces in the upper, zone of active slope movements, or increasing the resistance to shearing of the lower, zone of passive slope movements. The former proposal requires drainage, combined with the excavation of slopes above the head and upper lateral boundaries of the earthflow. The removal of such slopes would eliminate the source of debris being deposited in the head of the landslide through secondary landsliding, and hence eliminate the gravitational driving forces caused by the accumulation of the debris. While removal of all slopes contributing material to the head of the earthflow would prove to be prohibitive economically, and impractical from an engineering viewpoint, excavation of one of the more critical sites adjacent to the landslide could be feasible.

Subsurface drainage of the lower (foot) areas of the earthflow will be the only stabilization measures applicable to increasing the resistance to shearing of the zone of passive slope movements.

Surface ditches constructed by Railways have been draining the lower half of the earthflow for approximately three years. No increase in slope stability has so far accompanied the installation of these drains. However, properly constructed and maintained surface ditches are to be encouraged, as these promote rapid runoff and removal of water from the landslide surface.

Complete stabilization of the earthflow will prove exceedingly difficult to effect. Several correction measures (buttressing, slope treatment) have been found to be unsuited to this particular class of landslide (section 5.23, 5.25). Similarly, several subsurface drainage methods (horizontal drains, uncased vertical sand drains, drainage trenches) have also shown to be unsuited (section 5.24.3).

Recommendations include the following:

(a) A comprehensive slope regrading programme over the whole landslide surface is suggested. The programme should include the draining of all ponds and swampy ground, levelling of hummocky topography, and the sealing of all surface exposures of the shear zones defining the lateral boundaries of the earthflow. In addition, surface drains presently draining the foot of the landslide which cross the shear zones should be eliminated and replaced towards the centre of the earthflow.

Slope regrading is designed to promote rapid runoff of surface water and prevent infiltration of water into the slide mass through cracks and depressions. Every effort should be made towards these ends. It is recommended the slope regrading programme be undertaken during summer, when dryer weather should allow more favourable conditions for plant and machinery.

(b) Surface ditches presently draining the foot of the earthflow which run across the slope should be eliminated. The drains have insufficient gradient for rapid runoff of surface water.

(c) In conjunction with the slope regrading programme, it is recommended that the present scheme of installing surface drainage ditches be extended. Ditches will need to be constructed in the direction of movement, though lining the drains will not be necessary. In the lower half of the site, two to three ditches could satisfactorily be installed across the slope. In the upper part of the landslide, a single ditch would only be practical.

(d) Revegetation of the earthflow furface must be undertaken on completion of the slope regrading programme and installation of surface drains. Revegetation will initially consist of establishing a grass cover. Consideration should be given to establishing a tree cover over the whole site and on slopes adjacent to the landslide. This will aid principally in the removal of moisture through transpiration.

(e) Infinite slope stability analyses (section 4.73) indicate that the groundwater level in the slide mass will need to be lowered to at least half that of the present to significantly increase the stability of the slope. The reduced groundwater level will need to be approximately 9m, 30m, and 13m below the surface in boreholes A, B and C, respectively. Subsurface drainage will therefore be required to effect this reduced groundwater level.

A number of counterfort drains spaced at intervals across the slope, or a drainage curtain comprising a series of slotted, steel-lined shafts, are the only practical subsurface drainage schemes from an engineering viewpoint (section 5.24.3). However, the effectiveness of counterfort drains is limited by the depth to which plant can excavate; construction difficulties are usually experienced when this depth exceeds 5-6m. Comparing the depths to which groundwater will need to be lowered below the surface to effect significant safety factor increases, with a probable 6m depth limitation of counterfort drains, it is recommended that a counterfort drainage scheme is not warranted when only insignificant increases in slope stability will be the result.

A drainage curtain comprising a series of cased vertical shafts aligned across the slope could be considered in a region midway between the head and toe of the landslide. The principal advantage of installing the drainage curtain at this location would be to intercept water infiltrating from the head of the site and hence increase the shearing resistance of the soil mass in the lower (foot) region of the earthflow. Drains would need to be lined with perforated steel casing in order to prevent the disruption of the drainage curtain through earth movements. However, the installation of a drainage curtain would necessitate continuous submersible pumping (displacements at the bentonite-colluvium interface would prevent gravity flowage of water into a collector drain located beneath the slide surface); thus costs associated with continuous pumping of vertical drains would likely prove to be high.

(f) An engineering and economic feasibility study relating to the excavation of an area of secondary landsliding in a section of the ridge paralleling the southeast boundary of the earthflow is recommended (Fig. 26). At this site, translational sliding of bentonites and montmorillonitic siltstones towards the head of the landslide occurs (section 4.43).

Elimination of this area of landsliding would probably lead to a partial reduction in the gravitational driving forces within the upper, zone of active slope movements. A decrease in the magnitude of displacements lower in the slope could accompany a reduction of these driving forces.

The stability of the new slopes formed by the excavation should be critically examined to ensure that further landsliding into the earthflow would not occur.

The area involved in the secondary landsliding required to be excavated covers approximately one hectare. The volume of material required to be excavated should be determined through subsurface site investigations to ascertain the depth of sliding.

(g) Excavation of slopes above the inland margin of the railway should be avoided. Serious ground heaving (up to 0.4m) adjacent to, though not affecting, the track, followed lightening of these slopes during the installation of surface drains in July, 1976.

(h) Elimination of the landslide by bridging the railway across the fore of the earthflow will likely be the only long-term, permanent solution to the slope stability problems at Mikonui. However, present costs for a 200m long bridge sufficient to span the width of the toe are between N.Z. $0.75-1.0m. Bridging the earthflow is therefore likely to prove economically prohibitive.

SECTION 6: REFERENCES

AKROYD, T.N.W. Laboratory testing in soil engineering, Geotechnical Monograph No. 1., Soil Mechanics Ltd.

BAILEY, W. A., CHRISTIAN, J. T. (1969), ICES LEASE-1. A problem-oriented language for slope stability analysis. User's manual. Soil Mech. Div. & Civil Engng. Sys. Lab., Dept. Civil Engng., M.I.T.

BAKER, R. F. (1960). Prevention and correction of landslides, Section 11 in Highway Engineering Handbook. (Woods, K. B. editor.) McGraw Hill.

BAKER, R. F., MARSHALL, H. C. (1957). Control and correction of landslides. Highway Research Board Spec. Rep. No. 29.

BELL, D. H. (1977). Railway and highway stability problems in the Kaikoura area. N.Z. Inst. Engrs. Ann. Conf. (Christchurch), Paper No. 91.

BELL, D. H. (in press). High intensity rainstorms and geological hazards: Cyclone Alison, March 1975, Kaikoura, New Zealand. Bull. Int. Ass. Engng. Geol., No. 14.

BISHOP, A. W. (1955). The use of the slip circle in the stability analysis of slopes. Geotechnique, 1., No. 7.

BISHOP, A. W., MORGENSTERN, N. R. (1960). Stability coefficients for earth slopes. Geotechnique, 10., pp. 129-150.

BORRIE, G. (1977). Personal communication.

BROWN, I. R. (1975). The stability of slopes in soft rocks. Report No. 112. School of Engineering, University of Auckland.

BRUKL, E., SCHEIDEGGAR, A. (1973). Application of theory of plasticity to slow mudflows. Geotechnique, 23., No. 1.

CASHMAN, P. M. (1973). Groundwater control. Ground Engineering, 6., No. 5., pp. 26-29.

CEDERGREN, H. R. (1967). Seepage, drainage and flow nets. John Wiley and Sons, New York.

CHANDLER, R. J. (1974). Lias clay: the long-term stability of cutting slopes. Geotechnique, 24., No. 1., pp. 21-38.

CHANDLER, R. J., SKEMPTON, A. W. (1974). The design of permanent slopes in stiff fissured clays. Geotechnique, 24., No. 4., pp. 457-466.

CROZIER, M. J. (1969). Earthflow occurrence during high intensity rainfall in Eastern Otago (New Zealand). Eng. Geol. 3., pp. 325-334.

D'APPOLONIA, E., ALPERSTEIN, R., D'APPOLONIA, D. J. (1967). Behaviour of a colluvial slope. Jl. Soil Mech. & Found. Div., A.S.C.E. 93, SM 4., pp. 447-473.

EAST, G.R.W. (1974). Inclined plane slope failures in the Auckland Waitemata soils. N.Z. Inst. Engrs. Proc. Tech. Groups Vol. 1., Issue 5(G)., pp. 5.17-5.34.

EIGENBROD, K. D. (1975). Analysis of the pore pressure changes following the excavation of a slope. Can. Geotech. Jl., Vol. 12 (3)., pp. 429-440.

FETZER, C. A. (1967). Electro-osmotic stabilization of West Branch Dam. Jl. Soil Mech. & Found. Div., A.S.C.E., 93., SM 4.

FYFE, H. E. (1934). Bentonite and its occurrence in New Zealand. N.Z. Jl. Sci. Tech., Vol. 15 (6)., pp. 386-394.

FRANKLIN, D., CHANDLER, R. J. (1972). The slake durability test. Int. Jl. Rock Mech. Min. Sci., Vol. 9., pp. 325-341.

FREDLUND, D. G. (1975). Engineering properties of expansive clays. Transportation and Geotechnical Group, Dept. Civil Engng., University of Saskatchewan, Saskatoon, Saskatchewan.

Geological Society Engineering Group Working Party Report (1972). The preparation of maps and plans in terms of engineering geology. Q. Jl. Engng. Geol. 5 (4)., pp. 293-381.

GRAY, D. H. (1974). Reinforcement and stabilisation of soil by vegetation. Jl. Geotech. Engng. Div., A.S.C.E., 100., GT 6., pp. 695-698.

GREGG, D. R. (1964). Sheet 18 - Hurunui. Geological Map of New Zealand., 1:250,000., N.Z. Dept. Sci. Ind. Res., Wellington.

GROCOTT, G. G. (1975). An engineering geological appraisal of the stability problems at the Hawkswood Cutting, North Canterbury. Unpublished report to the N.Z. Railways Dept.

GROCOTT, G. G., HUNTER, G. (1975). Slope stability investigations at the Hawkswood railway cutting, North Canterbury. Abstracts, N.Z. Geol. Soc. Conf., Kaikoura.

HAEFELI, R. (1948). The stability of slopes acted upon by parallel seepage. Proc. 2nd Int. Conf. Soil Mech. (Rotterdam) 1, pp. 57-62.

HAWKINS, L. V. (1961). The reciprocal method of routine shallow seismic refraction investigations. Geophysics, 26., No. 6., pp. 806-819.

HAWLEY, J. G., RIDDOLLS, B. W., BARKER, P. R. (1975). Soft rock engineering in the Central North Island of New Zealand. Proc. 2nd Aust. N.Z. Geomech. Conf. (Brisbane)., pp. 201-206.

HENKE, K. F. (1970). Clearance of slides by the application of horizontal draining boreholes and electro-osmosis (in German). Bauingenieur 45., No. 7., pp. 235-241.

HOEK, E., BRAY, J. W. (1974). Rock slope engineering. Institution of Mining and Metallurgy, London.

HUNTER, J. H., SCHUSTER, R. L. (1968). Stability of simple cuttings in normally consolidated clays. Geotechnique, 18., No. 3., pp. 372-378.

HUTCHINSON, J. N. (1968). Mass movement. In Encyclopaedia of Geomorphology. (Fairbridge, F. W. editor), Reinhold Book Corporation, New York.

HUTCHINSON, J. (1970). A coastal mudflow on the London clay cliffs at Beltringe, north Kent. Geotechnique, 20., No. 4., pp. 412-438.

ISDALE, H. A. (1965). Christchurch-Picton Railway - study in development and functional change. MA thesis (Geography), University of Canterbury, Christchurch.

JANBU, N. (1973). Slope stability computations. Embankment dam engineering, Casagrande volume. Hirschfield, R.C., Poulos, S. J. editors.) pp. 47-86.

KAYES, T. J. (1974). Analysis of natural earth slope stability. N.Z. Inst. Engrs. Proc. Tech. Groups Vol. 1., Issue 5(G)., pp. 6.13-6.23.

KREY, H. (1926). Erddruck, Erdwiderstand und Tragiahigkeit des Baugrundes. Berlin, Ernst.

KRYNINE, D. P., JUDD, R. J. (1957). Principles of engineering geology and geotechnics. McGraw Hill.

McLEAN, J. D. (1976). A systematic method of soil description for engineering soils use. Unpublished N.Z. Geological Survey report., Twizel.

MAHTAB, M. A., GOODMAN, R. E. (1970). Three dimensional finite element analysis of jointed rock slopes. Proc. 2nd Conf. Int. Soc. Rock Mech. 7., pp. 353.

MANCINI, P. (1971). The electro-chemical stabilization of a landslide (in Italian). Riv. Ital. Geotecn. 5., No. 4 bis., pp. 297-306.

MANSUR, C. I., KAUFMAN, R. I. (1962). Dewatering, Chapter 3 in Foundation Engineering. (Leonards, G. editor.) McGraw Hill.

MAXWELL, P. A. (1964). Structural geology and pre-Quaternary stratigraphy, Kaiwara, North Canterbury. MSc thesis (Geology), University of Canterbury, Christchurch.

MERRIAM, R. (1960). Portugese Bend landslide, Palos Verdes Hills, California. Jl. Geol. 68., pp. 140-153.

MORGENSTERN, N. R., PRICE, V. E. (1965). The analysis of the stability of general slip surfaces. Geotechnique, 15., pp. 79-93.

MYSLIVEC, A. (1969). New methods of landslide correction (in Czechoslovakian). Inz. Stavby 17., No. 5., pp. 193-169.

New Zealand Economist. (August, 1976). Do we need the Railways? Vol. 38., No. 5., pp. 20-37.

N.Z. Geological Survey (1973)., Quaternary Geology - South Island., 1:1,000,000., (1st edition)., N.Z. Geological Survey Miscellaneous Series, Map 6., Dept. Sci. Ind. Res., Wellington.

New Zealand Standard 4402P., Part 1., (1976). Provincial New Zealand standard. Methods of testing soils for civil engineering purposes. Soil classification and chemical tests.

NOBLE, H. L. (1973). Residual strength and landslides in clay and shale. Jl. Soil Mech. & Found. Div., A.S.C.E., 99, SM 9., pp. 705.

NORTHEY, R. D., HAWLEY, J. G., BARKER, P. R. (1974). Classification and mechanisms of slope failures in natural ground. N.Z. Inst. Engrs. Proc. Tech. Groups Vol. 1., Issue 5(G)., pp. 3.1-3.8.

OLSEN, A. J. (1974). Discussion on paper by Northey et al (1974), N.Z. Inst. Engrs. Proc. Tech. Groups Vol. 1., Issue 5(G)., pp. 3.11.

PALMER, L. A., THOMPSON, J. B., YEOMANS, C. M. (1950). The control of a landslide by subsurface drainage. Highway Research Board Proc. Vol. 30., pp. 503.

PARROTT, W. T. (1955). Control of a landslide by vertical sand drains. Highway Research Board Bull. 115., pp. 51.

PARTON, I. M. (1974). Chief designing engineer (civil) on North Island Main Trunk Railway, Poro-o-Tarao Tunnel replacement. South portal approach and structure, design report. Special projects design office, Ministry of Works and Development, Wellington.

PARTON, I. M. (1974). Investigations at the site of the Poro-o-Tarao Tunnel., Central Laboratories report No. 473., Ministry of Works and Development.

PATERSON, W.S.B. (1972). The physics of glaciers. Pergamon Press.

PDP-8 Programming Manual. Digital Equipment Corporation, Maynard, Massachusetts, 1970.

PETRIE, J. M. (1974). Chaotic Mesozoic rocks of the Studleigh Range, North Canterbury. MSc thesis (Geology), University of Canterbury, Christchurch.

Rails. (December, 1976.) New Zealand's rail transport journal, pp.8-9.

SCHUSTER, R. L. (1968). Selection of analytical methods and strength parameters for slope stability investigations in cohesive soils. In Highway Research Board, Highway Research Record No. 223., Embankment Analysis, Settlement, and Seepage., Washington, D.C.

SCHWAKOWSKI, G. (1971). Inclinometer measurements and their importance in the analysis of slope stability (in German). Zeit. fur Ange. Geol, Bd. 17., Heft 4., pp. 134-138.

SEED, H. B., WILSON, S. D. (1967). The Turnagain Heights landslide, Anchorage, Alaska. Jl. Soil Mech. & Found. Div., A.S.C.E. 93, SM 4., pp. 325-353.

SHARPE, C.F.S. (1938). Landslides and related phenomena. Columbia University Press, New York Press.

SKEMPTON, A. W. (1964). Long-term stability of clay slopes. Geotechnique, 14., pp. 77-101.

SKEMPTON, A. W. (1970). First time slides in overconsolidated clays. Geotechnique, 20., No. 3., pp. 320-324.

SKEMPTON, A. W., HUTCHINSON, J. (1969). Stability of natural slopes and embankment foundations. Proc. 7th Int. Conf. Soil Mech. Found. Engng. (Mexico), pp. 291-340.

SMITH, N. S. (1974). A contractors viewpoint and recommendations. N.Z. Inst. Engrs. Proc. Tech. Groups Vol. 1., Issue 5(G)., pp. 5.1-5.16.

AQUIER, L. R., VERSTEEG, J. H. (1971). The history and correction of the OMSI-Zoo landslide. Engng. Geol. & Soils Symp., 9th., Proc., Idaho Dept. Highways., pp.237-256.

TERZAGHI, K., PECK, R. B. (1967). Soil mechanics in engineering practice., 2nd edition. John Wiley and Sons, New York.

University of Canterbury (1973). A guide to the presentation of theses, (Christchurch).

U.S. Dept. of the Interior. Bureau of Reclamation. A guide to the use of soils as foundations and as construction materials for hydraulic structures, 1st Edition - Revised 1968.

VARNES, D. J. (1958). Landslide types and processes. Landslide and Engineering Practise. Highway Research Board Spec. Rep. No. 20., pp. 20-47.

VAUGHAN, P. R., WALBANCKE, H. J. (1973). Pore pressure changes and the delayed failure of cutting slopes in over-consolidated clay. Geotechnique 23, No. 4., pp. 531-539.

VEDER, C. (1973). The phenomenon of contact zones of soil mechanics. Ground Engineering, 6., No. 5., pp. 30-38.

VERSTEEG, J. H.(1976). Personal communication.

VOIGHT, B. (1973). Correlation between Atterberg plasticity limits and residual strength of natural soils. Geotechnique, 23., No. 2., pp. 165.

WARREN, G. (1975). Personal communication.

__________ (1975). Geological sketch map produced for a field trip, Geol. Soc. N.Z. Conf., Kaikoura.

__________ (in prep.) A thesis undertaken for the degree of Doctor of Philosophy (Geology) in the University of Canterbury, Christchurch.

WARREN, G., SPEDEN, I. (1977 in prep.) The Piripauan and Haumurian Stratotypes (Mata Series, Upper Cretaceous) and correlative sequences in the Haumuri Bluff district, South Marlborough. N.Z. Geological Survey Bulletin No. 92.

ZARUBA, Q., MENCL, V. (1969). Landslides and their control. Elsevier, Amsterdam.

SECTION 7: APPENDICES

7.1 APPENDIX 1 - RESULTS OF SURFACE DEFORMATION MONITORING, ETHELTON SLIP. A LIST OF THE FORTRAN COMPUTER PROGRAMME IS ALSO INCLUDED.

********ETHELTON SLIP-NORTH CANTERBURY-NEW ZEALAND********

****CONTROL SURVEY-FIELD DATA REDUCTION****

DATE OF THIS SURVEY IS 30- 5-76

NUMBER OF MONTHS SINCE INITIAL SURVEY= 1.6

NUMBER OF CONTROL POINTS= 8

ALL READINGS FROM STATION 1

STATION 8 ASSUMES FIXED BEARING

SUMMARY OF RESULTS FROM INITIAL SURVEY OF 11-4-76

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.357	6500.439	90.971	102.	10.	18.70	256.682	-40.346
3	-22229.850	6436.073	91.950	72.	37.	11.00	195.469	-39.367
4	-22176.851	6408.703	94.035	55.	0.	59.50	194.278	-37.282
5	-22035.307	6317.558	93.666	15.	3.	13.20	261.920	-37.651
6	-22847.932	6814.541	88.566	134.	43.	43.90	795.297	-42.751
7	-22096.945	6497.707	146.990	52.	22.	32.70	313.346	15.673
8	-21844.362	7337.502	406.721	67.	48.	19.13	1175.038	275.404

SUMMARY OF AVERAGE VALUES OF INPUT DATA FROM THIS SURVEY

STATION NUMBER	AVERAGE BEARING DIFFERENCE FROM FIXED BEARING STATION DEGS	MINS	SECS	AVERAGE VERTICAL ANGLE DEGS	MINS	SECS	AVERAGE SLOPE DISTANCE
1	NOT APPLICABLE			NOT APPLICABLE			NOT APPLICABLE
2	-34	-22	-1.	-8	-56	-14.	259.845
3	-4	-48	-60.	-11	-25	-48.	199.410
4	12	47	12.	-10	-54	-48.	197.843
5	52	45	2.	-8	-13	-16.	264.628
6	-66	-55	-25.	-3	-5	-16.	796.452
7	15	25	47.	2	49	36.	313.727
8	0	0	0.	13	11	26.	1206.853

SUMMARY OF RESULTS FROM THIS SURVEY

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.357	6500.429	90.950	102.	10.	20.	256.671	-40.367
3	-22229.859	6436.067	91.949	72.	37.	19.	195.461	-39.368
4	-22176.861	6408.701	94.010	55.	1.	7.	194.270	-37.307
5	-22035.315	6317.557	93.627	15.	3.	17.	261.912	-37.690
6	-22847.930	6814.543	88.566	134.	43.	44.	795.297	-42.751
7	-22096.947	6497.706	146.938	52.	22.	33.	313.344	15.621
8	-21844.362	7337.502	406.721	67.	48.	19.	1175.038	275.404

CALCULATED MOVEMENTS

STATION NUMBER	CHANGE IN COORDINATES NORTH	EAST	CHANGE IN REDUCED LEVEL	MAGNITUDE OF MOVEMENT	MOVEMENT PER MONTH	DIRECTION OF MOVEMENT IN DEGREES BEARING	ELEVATION
1	0.000	0.000	0.000	0.000	0.00000		
2	-0.000	-0.010	-0.021	0.024	0.01449	269.000	-63.756
3	-0.009	-0.006	-0.001	0.011	0.00664	215.734	-4.885
4	-0.010	-0.002	-0.025	0.027	0.01668	192.528	-68.057
5	-0.008	-0.001	-0.039	0.040	0.02445	184.986	-78.720
6	0.002	0.002	0.000	0.003	0.00166	52.110	3.433
7	-0.002	-0.001	-0.052	0.052	0.03185	200.676	-87.135
8	0.000	0.000	0.000	0.001	0.00034	40.928	16.219

**********CALCULATIONS COMPLETE**********

********ETHELTON SLIP-NORTH CANTERBURY-NEW ZEALAND********

****CONTROL SURVEY-FIELD DATA REDUCTION****

DATE OF THIS SURVEY IS 2- 8-76

NUMBER OF MONTHS SINCE INITIAL SURVEY= 3.7

NUMBER OF CONTROL POINTS= 8

ALL READINGS FROM STATION 1

STATION 8 ASSUMES FIXED BEARING

SUMMARY OF RESULTS FROM INITIAL SURVEY OF 11-4-76

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.357	6500.439	90.971	102.	10.	18.70	256.682	-40.346
3	-22229.850	6436.073	91.950	72.	37.	11.00	195.469	-39.367
4	-22176.851	6408.703	94.035	55.	0.	59.50	194.278	-37.282
5	-22035.307	6317.558	93.666	15.	3.	13.20	261.920	-37.651
6	-22847.932	6814.541	88.566	134.	43.	43.90	795.297	-42.751
7	-22096.945	6497.707	146.990	52.	22.	32.70	313.346	15.673
8	-21844.362	7337.502	406.721	67.	48.	19.13	1175.038	275.404

SUMMARY OF AVERAGE VALUES OF INPUT DATA FROM THIS SURVEY

STATION NUMBER	AVERAGE BEARING DIFFERENCE FROM FIXED BEARING STATION DEGS	MINS	SECS	AVERAGE VERTICAL ANGLE DEGS	MINS	SECS	AVERAGE SLOPE DISTANCE
1	NOT APPLICABLE			NOT APPLICABLE			NOT APPLICABLE
2	-34	-21	-41.	-6	-58	-19.	259.843
3	-4	-48	-32.	-11	-25	-57.	199.417
4	12	47	34.	-10	-54	-54.	197.844
5	52	45	15.	-8	-13	-11.	264.630
6	-66	-55	-25.	-3	-5	-15.	796.452
7	15	25	54.	2	49	55.	313.721
8	0	0	0.	13	11	26.	1206.853

SUMMARY OF RESULTS FROM THIS SURVEY

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.333	6500.431	90.943	102.	10.	1.	256.668	-40.374
3	-22229.832	6436.064	91.939	72.	36.	52.	195.466	-39.378
4	-22176.844	6408.689	94.004	55.	0.	45.	194.270	-37.313
5	-22035.308	6317.543	93.632	15.	3.	4.	261.915	-37.685
6	-22847.931	6814.543	88.568	134.	43.	44.	795.297	-42.749
7	-22096.943	6497.693	146.967	52.	22.	25.	313.337	15.650
8	-21844.362	7337.502	406.721	67.	48.	19.	1175.038	275.404

CALCULATED MOVEMENTS

STATION NUMBER	CHANGE IN COORDINATES NORTH	EAST	CHANGE IN REDUCED LEVEL	MAGNITUDE OF MOVEMENT	MOVEMENT PER MONTH	DIRECTION OF MOVEMENT IN DEGREES BEARING	ELEVATION
1	0.000	0.000	0.000	0.000	0.00000		
2	0.024	-0.008	-0.028	0.038	0.01015	340.841	-47.372
3	0.018	-0.009	-0.011	0.023	0.00615	332.275	-29.243
4	0.007	-0.014	-0.031	0.035	0.00950	296.076	-63.214
5	-0.001	-0.015	-0.034	0.037	0.01000	266.640	-65.365
6	0.001	0.002	0.002	0.003	0.00079	55.306	35.798
7	0.002	-0.014	-0.023	0.027	0.00729	279.516	-58.248
8	0.000	0.000	0.000	0.001	0.00015	40.928	16.219

**********CALCULATIONS COMPLETE**********

********ETHELTON SLIP-NORTH CANTERBURY-NEW ZEALAND********

****CONTROL SURVEY-FIELD DATA REDUCTION****

DATE OF THIS SURVEY IS 29- 9-76

NUMBER OF MONTHS SINCE INITIAL SURVEY= 5.6

NUMBER OF CONTROL POINTS= 8

ALL READINGS FROM STATION 1

STATION 8 ASSUMES FIXED BEARING

SUMMARY OF RESULTS FROM INITIAL SURVEY OF 11-4-76

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.357	6500.439	90.971	102.	10.	18.70	256.682	-40.346
3	-22229.850	6436.073	91.950	72.	37.	11.00	195.469	-39.367
4	-22176.851	6408.703	94.035	55.	0.	59.50	194.278	-37.282
5	-22035.307	6317.558	93.666	15.	3.	13.20	261.920	-37.651
6	-22847.932	6814.541	88.566	134.	43.	43.90	795.297	-42.751
7	-22096.945	6497.707	146.990	52.	22.	32.70	313.346	15.673
8	-21844.362	7337.502	406.721	67.	48.	19.13	1175.038	275.404

SUMMARY OF AVERAGE VALUES OF INPUT DATA FROM THIS SURVEY

STATION NUMBER	AVERAGE BEARING DIFFERENCE FROM FIXED BEARING STATION DEGS	MINS	SECS	AVERAGE VERTICAL ANGLE DEGS	MINS	SECS	AVERAGE SLOPE DISTANCE
1	NOT APPLICABLE			NOT APPLICABLE			NOT APPLICABLE
2	-34	-21	-53.	-8	-58	-22.	259.843
3	-4	-48	-46.	-11	-26	-1.	199.414
4	12	47	36.	-10	-55	-30.	197.778
5	52	45	7.	-8	-13	-12.	264.627
6	-66	-55	-25.	-3	-5	-15.	796.452
7	15	25	50.	2	49	55.	313.722
8	0	0	0.	13	11	26.	1206.853

SUMMARY OF RESULTS FROM THIS SURVEY

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	BEARING MINS	BEARING SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.348	6500.427	90.939	102.	10.	13.	256.667	-40.378
3	-22229.846	6436.064	91.936	72.	37.	5.	195.462	-39.381
4	-22176.883	6408.629	93.982	55.	0.	43.	194.198	-37.335
5	-22035.314	6317.552	93.632	15.	3.	13.	261.912	-37.685
6	-22847.931	6814.543	88.568	134.	43.	44.	795.297	-42.749
7	-22096.947	6497.697	146.967	52.	22.	29.	313.338	15.650
8	-21844.362	7337.502	406.721	67.	48.	19.	1175.038	275.404

CALCULATED MOVEMENTS

STATION NUMBER	CHANGE IN COORDINATES NORTH	EAST	CHANGE IN REDUCED LEVEL	MAGNITUDE OF MOVEMENT	MOVEMENT PER MONTH	DIRECTION OF MOVEMENT IN DEGREES BEARING	ELEVATION
1	0.000	0.000	0.000	0.000	0.00000		
2	0.009	-0.012	-0.032	0.035	0.00628	307.869	-64.082
3	0.004	-0.009	-0.014	0.017	0.00305	294.020	-55.656
4	-0.032	-0.074	-0.053	0.096	0.01722	246.382	-33.298
5	-0.007	-0.006	-0.034	0.035	0.00626	221.966	-75.282
6	0.001	0.002	0.002	0.003	0.00052	55.306	35.798
7	-0.002	-0.010	-0.023	0.025	0.00451	260.192	-67.230
8	0.000	0.000	0.000	0.001	0.00010	40.926	16.219

**********CALCULATIONS COMPLETE**********

********ETHELTON SLIP-NORTH CANTERBURY-NEW ZEALAND********

****CONTROL SURVEY-FIELD DATA REDUCTION****

DATE OF THIS SURVEY IS 3- 2-77

NUMBER OF MONTHS SINCE INITIAL SURVEY= 9.7

NUMBER OF CONTROL POINTS= 8

ALL READINGS FROM STATION 1

STATION 8 ASSUMES FIXED BEARING

SUMMARY OF RESULTS FROM INITIAL SURVEY OF 11-4-76

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	MINS	SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.357	6500.439	90.971	102.	10.	18.70	256.682	-40.346
3	-22229.850	6436.073	91.950	72.	37.	11.00	195.469	-39.367
4	-22176.851	6408.703	94.035	55.	0.	59.50	194.276	-37.282
5	-22035.307	6317.558	93.666	15.	3.	13.20	261.920	-37.651
6	-22847.932	6814.541	88.566	134.	43.	43.90	795.297	-42.751
7	-22096.945	6497.707	146.990	52.	22.	32.70	313.346	15.673
8	-21844.362	7337.502	406.721	67.	48.	19.13	1175.038	275.404

SUMMARY OF AVERAGE VALUES OF INPUT DATA FROM THIS SURVEY

STATION NUMBER	AVERAGE BEARING DIFFERENCE FROM FIXED BEARING STATION DEGS	MINS	SECS	AVERAGE VERTICAL ANGLE DEGS	MINS	SECS	AVERAGE SLOPE DISTANCE
1	NOT APPLICABLE			NOT APPLICABLE			NOT APPLICABLE
2	-34	-21	-51.	-8	-58	-21.	259.845
3	-4	-48	-52.	-11	-25	-60.	199.402
4	12	47	44.	-10	-55	-36.	197.739
5	52	45	13.	-8	-13	-7.	264.621
6	-66	-55	-25.	-3	-5	-15.	796.452
7	15	25	49.	2	49	58.	313.710
8	0	0	0.	13	11	26.	1206.853

SUMMARY OF RESULTS FROM THIS SURVEY

STATION NUMBER	NORTH COORDINATE	EAST COORDINATE	REDUCED LEVEL	BEARING DEGS	MINS	SECS	HORIZONTAL DISTANCE	VERTICAL DISTANCE
1	-22288.238	6249.528	131.317					
2	-22342.345	6500.430	90.941	102.	10.	10.	256.670	-40.376
3	-22229.854	6436.055	91.939	72.	37.	11.	195.450	-39.378
4	-22176.900	6408.593	93.985	55.	0.	35.	194.159	-37.332
5	-22035.316	6317.543	93.638	15.	3.	6.	261.907	-37.679
6	-22847.930	6814.543	88.568	134.	43.	44.	795.297	-42.749
7	-22096.956	6497.689	146.971	52.	22.	30.	313.325	15.654
8	-21844.362	7337.502	406.721	67.	48.	19.	1175.038	275.404

CALCULATED MOVEMENTS

STATION NUMBER	CHANGE IN COORDINATES NORTH	EAST	CHANGE IN REDUCED LEVEL	MAGNITUDE OF MOVEMENT	MOVEMENT PER MONTH	DIRECTION OF MOVEMENT IN DEGREES BEARING	ELEVATION
1	0.000	0.000	0.000	0.000	0.00000		
2	0.012	-0.009	-0.030	0.034	0.00345	321.620	-63.481
3	-0.004	-0.018	-0.011	0.022	0.00224	257.012	-30.140
4	-0.049	-0.110	-0.050	0.131	0.01342	246.104	-22.716
5	-0.009	-0.015	-0.028	0.033	0.00339	238.505	-56.868
6	0.002	0.002	0.002	0.003	0.00033	53.831	31.856
7	-0.011	-0.018	-0.019	0.028	0.00292	240.050	-42.187
8	0.000	0.000	0.000	0.001	0.00006	40.926	16.219

**********CALCULATIONS COMPLETE**********

```
C       CONTROL SURVEY REDUCTION FOR ETHELTON SLIP                              C  000:0000:5
                                                                                   START OF SEGMENT 002
                                                                                FORMAT SEGMENT IS 00CA LONG
                                                                                FORMAT SEGMENT IS 00CC LONG
        DIMENSION COORIN(20),COORIE(20),RLI(20),VDI(20),BEARDI(20)              C  002:0000:0
        DIMENSION BEARMI(20),BEARSI(20),DISTI(20),ZD(20,4),ZM(20,4)             C  002:0000:0
        DIMENSION ZS(20,4),VAD(20,4),VAM(20,4),VAS(20,4),SDIST(20)              C  002:0000:0
        DIMENSION BEARRI(20),THETAR(20,4),AVDIFF(20),AVDDF(20),NAVDD(20)        C  002:0000:0
        DIMENSION AVDDW(20),AVDMF(20),NAVDM(20),AVDMW(20),AVDSF(20)             C  002:0000:0
        DIMENSION ACBEAR(20),BEARDF(20),NBEARD(20),BEARDW(20),BEARMF(20)        C  002:0000:0
        DIMENSION NBEARM(20),BEARMW(20),BEARSF(20),VARD4(20,4),VAAVR(20)        C  002:0000:0
        DIMENSION VADAVF(20),NVADAV(20),VADAVW(20),VAMAVF(20),NVAMAV(20)        C  002:0000:0
        DIMENSION VAMAVW(20),VASAVF(20),C2(20),FSQRT(20),HORDIS(20)             C  002:0000:0
        DIMENSION VERDIS(20),COORFN(20),COORFE(20),RLF(20),DELTAN(20)           C  002:0000:0
        DIMENSION DELTAE(20),DELTAR(20),DELTAT(20),RATEMM(20),BMID(20)          C  002:0000:0
        DIMENSION ANGELM(20)                                                    C  002:0000:0
C                                                                               C  002:0000:0
C       PROGRAM HEADING, DATE AND MAIN CONTROL POINTS                           C  002:0000:0
        READ(5,10)IDNOW,ICMNOW,IYNOW                                            C  002:0000:0
                                                                                      FIB IS 0006 LONG
     10 FORMAT(3I2)                                                             C  002:000B:2
        READ(5,20)N,NSET,NFIX                                                   C  002:000B:2
     20 FORMAT(3I2)                                                             C  002:0016:2
        WRITE(6,30)                                                             C  002:0016:2
                                                                                      FIB IS 0006 LONG
     30 FORMAT('1',36X,'********ETHELTON SLIP-NORTH CANTERBURY-NEW ZEALAND      C  002:001A:2
       /********')                                                              C  002:001A:2
        WRITE(6,40)                                                             C  002:001A:2
     40 FORMAT('0',44X,'****CONTROL SURVEY-FIELD DATA REDUCTION****')           C  002:001E:2
        WRITE(6,50)IDNOW,ICMNOW,IYNOW                                           C  002:001E:2
     50 FORMAT('0',50X,'DATE OF THIS SURVEY IS',1X,I2,'-',I2,                   C  002:0028:2
       /'-',I2)                                                                 C  002:0028:2
        DNOW=IDNOW                                                              C  002:0028:2
        CMNOW=ICMNOW                                                            C  002:0029:1
        YNOW=IYNOW                                                              C  002:002A:0
        TSIRIM=((YNOW-76.)*12.+(CMNOW-4.)+(DNOW-11.)/30.)                       C  002:002A:5
        WRITE(6,60)TSIRIM                                                       C  002:002E:5
     60 FORMAT('0',44X,'NUMBER OF MONTHS SINCE INITIAL SURVEY=',F5.1)           C  002:0035:2
        WRITE(6,70)N                                                            C  002:0035:2
     70 FORMAT('0',52X,'NUMBER OF CONTROL POINTS=',I2)                          C  002:003C:2
        WRITE(6,80)NSET                                                         C  002:003C:2
     80 FORMAT('0',52X,'ALL READINGS FROM STATION',I2)                          C  002:0043:2
        WRITE(6,90)NFIX                                                         C  002:0043:2
     90 FORMAT('0',50X,'STATION',1X,I2,1X,'ASSUMES FIXED BEARING')              C  002:004A:2
C                                                                               C  002:004A:2
C       DATA FROM INITIAL SURVEY OF 11-4-76                                     C  002:004A:2
        AN=N                                                                    C  002:004A:2
        AN1=AN/2.                                                               C  002:004B:1
        N1=AN1                                                                  C  002:004C:1
        N2=N1+1                                                                 C  002:004C:5
        READ(5,100)(COORIN(I),I=1,N1)                                           C  002:004D:5
        READ(5,100)(COORIN(I),I=N2,N)                                           C  002:005A:2
        READ(5,100)(COORIE(I),I=1,N1)                                           C  002:0067:2
        READ(5,100)(COORIE(I),I=N2,N)                                           C  002:0074:2
    100 FORMAT(4F10.3)                                                          C  002:0081:2
        READ(5,110) (RLI(I),I=1,N)                                              C  002:0081:2
    110 FORMAT(8F7.3)                                                           C  002:008E:2
        DO 115 I=1,N                                                            C  002:008E:2
    115 VDI(I)=RLI(I)-RLI(NSET)                                                 C  002:008F:0
        DO 130 I=1,N                                                            C  002:0094:4
        IF(I.EQ.NSET)GO TO 130                                                  C  002:0096:0
        READ(5,120) BEARDI(I)                                                   C  002:0097:2
    120 FORMAT(F4.0)                                                            C  002:009F:2
    130 CONTINUE                                                                C  002:009F:2
        DO 140 I=1,N                                                            C  002:00A1:3
        IF(I.EQ.NSET)GO TO 140                                                  C  002:00A3:0
        READ(5,135) BEARMI(I)                                                   C  002:00A4:2
    135 FORMAT(F3.0)                                                            C  002:00AC:2
    140 CONTINUE                                                                C  002:00AC:2
        DO 150 I=1,N                                                            C  002:00AE:3
        IF(I.EQ.NSET)GO TO 150                                                  C  002:00B0:0
        READ(5,145) BEARSI(I)                                                   C  002:00B1:2
    145 FORMAT(F5.2)                                                            C  002:00B9:2
    150 CONTINUE                                                                C  002:00B9:2
        DO 160 I=1,N                                                            C  002:00BB:3
```

```
      IF(I.EQ.NSET)GO TO 160                                          C  002:00BD:0
      READ(5,155) DISTI(I)                                            C  002:00BE:2
  155 FORMAT(F8.3)                                                    C  002:00C6:2
  160 CONTINUE                                                        C  002:00C6:2
      READ(5,390)HT,HTT,HD,HDT,HTST                                   C  002:00C8:3
      WRITE(6,170)                                                    C  002:00D7:2
  170 FORMAT("0",/,/,41X,"SUMMARY OF RESULTS FROM INITIAL SURVEY OF 11-4  C  002:00DB:2
     /-76")                                                           C  002:00DB:2
      WRITE(6,180)                                                    C  002:00DB:2
  180 FORMAT('0','STATION',12X,'NORTH',17X,'EAST',14X,'REDUCED',15X,  C  002:00DF:2
     /'BEARING',16X,'HORIZONTAL',10X,'VERTICAL')                      C  002:00DF:2
      WRITE(6,190)                                                    C  002:00DF:2
  190 FORMAT(' ','NUMBER',11X,'COORDINATE',11X,'COORDINATE',12X,'LEVEL',  C  002:00E3:2
     /12X,'DEGS',2X,'MINS',2X,'SECS',12X,'DISTANCE',11X,'DISTANCE')   C  002:00E3:2
      DO 208 I=1,N                                                    C  002:00E3:2
      IF(I.EQ.NSET)GO TO 204                                          C  002:00E4:0
      WRITE(6,200)I,COORIN(I),COORIE(I),RLI(I),BEARDI(I),             C  002:00E5:2
     /BEARMI(I),BEARSI(I),DISTI(I),VDI(I)                             C  002:00F2:0
  200 FORMAT(' ',2X,I2,13X,F10.3,12X,F8.3,12X,F7.3,11X,F4.0,2X,       C  002:00FD:2
     /F3.0,2X,F5.2,12X,F8.3,11X,F7.3)                                 C  002:00FD:2
      GO TO 208                                                       C  002:00FD:2
  204 WRITE(6,206)I,COORIN(I),COORIE(I),RLI(I)                        C  002:00FD:5
  206 FORMAT(' ',2X,I2,13X,F10.3,12X,F8.3,12X,F7.3)                   C  002:010B:2
  208 CONTINUE                                                        C  002:010B:2
C                                                                     C  002:010D:3
C     INPUT DATA AND CALCULATION OF THEIR AVERAGE VALUES              C  002:010D:3
      DO 220 J1=1,N                                                   C  002:010D:3
      IF(J1.EQ.NSET)GO TO 220                                         C  002:010F:0
      DO 209 J2=1,4                                                   C  002:0110:2
  209 READ(5,210)ZD(J1,J2),ZM(J1,J2),ZS(J1,J2)                        C  002:0111:0
  210 FORMAT(  F4.0,F3.0,F4.1)                                        C  002:0124:3
  220 CONTINUE                                                        C  002:0124:3
      DO 240 J3=1,N                                                   C  002:0126:4
      IF(J3.EQ.NSET)GO TO 240                                         C  002:0128:0
      DO 225 J4=1,4                                                   C  002:0129:2
  225 READ(5,230)VAD(J3,J4),VAM(J3,J4),VAS(J3,J4)                     C  002:012A:0
  230 FORMAT(F4.0,F3.0,F4.1)                                          C  002:013D:3
  240 CONTINUE                                                        C  002:013D:3
      DO 255 I=1,N                                                    C  002:013F:4
      IF(I.EQ.NSET)GO TO 255                                          C  002:0141:0
      READ(5,250) SDIST(I)                                            C  002:0142:2
  250 FORMAT(F8.3)                                                    C  002:014A:2
  255 CONTINUE                                                        C  002:014A:2
      A=(ATAN(1.))/45.                                                C  002:014C:3
      DO 260 I=1,N                                                    C  002:014E:2
      IF(I.EQ.NSET) GO TO 260                                         C  002:014F:0
      BEARRI(I)=A*(BEARDI(I)+BEARMI(I)/60.+BEARSI(I)/3600.)           C  002:0150:2
  260 CONTINUE                                                        C  002:0156:4
      DO 280 J1=1,N                                                   C  002:0158:5
      IF(J1.EQ.NSET)GO TO 280                                         C  002:015A:0
      DO 270 J2=1,4                                                   C  002:015B:2
  270 THETAR(J1,J2)=A*(ZD(J1,J2)+ZM(J1,J2)/60.+ZS(J1,J2)/3600.)       C  002:015C:0
  280 CONTINUE                                                        C  002:0169:1
C     BEARING DIFFERENCE=AVE VALUE(FIXED BEARING-BEARING(I))          C  002:016B:2
      DO 300 J1=1,N                                                   C  002:016B:2
      IF(J1.EQ.NSET)GO TO 300                                         C  002:016C:0
      IF(BEARRI(J1).GT.BEARRI(NFIX)) GO TO 290                        C  002:016D:2
      DO 285 J2=1,4                                                   C  002:0170:0
      IF(THETAR(J1,J2).GT.THETAR(NFIX,J2)) THETAR(J1,J2)=THETAR(J1,J2)-  C  002:0171:0
     ?8.*ATAN(1.)                                                     C  002:0178:4
  285 AVDIFF(J1)=AVDIFF(J1)+(THETAR(NFIX,J2)-THETAR(J1,J2))/4.        C  002:017A:2
      GO TO 300                                                       C  002:0183:2
  290 DO 295 J2=1,4                                                   C  002:0183:5
      IF(THETAR(J1,J2).LT.THETAR(NFIX,J2)) THETAR(J1,J2)=THETAR(J1,J2)  C  002:0185:0
     ?+8.*ATAN(1.)                                                    C  002:018C:4
  295 AVDIFF(J1)=AVDIFF(J1)+(THETAR(NFIX,J2)-THETAR(J1,J2))/4.        C  002:018E:2
  300 CONTINUE                                                        C  002:0197:2
      DO 315 I=1,N                                                    C  002:0199:3
      IF(I.EQ.NSET)GO TO 315                                          C  002:019B:0
      AVDDF(I)=AVDIFF(I)/A                                            C  002:019C:2
      NAVDD(I)=AVDDF(I)                                               C  002:019F:1
      AVDDW(I)=NAVDD(I)                                               C  002:01A1:4
      AVDMF(I)=(AVDDF(I)-AVDDW(I))*60.                                C  002:01A4:0
      NAVDM(I)=AVDMF(I)                                               C  002:01A8:0
      AVDMW(I)=NAVDM(I)                                               C  002:01AA:3
  310 AVDSF(I)=(AVDMF(I)-AVDMW(I))*60.                                C  002:01AC:5
  315 CONTINUE                                                        C  002:01B0:5
```

```
      DO 335 I=1,N                                                      C    002:01B3:0
      IF(I.EQ.NSET)GO TO 335                                            C    002:01B4:0
      DO 320 J=1,4                                                      C    002:01B5:2
      IF(VAD(I,J).LT.180.) GO TO 318                                    C    002:01B6:0
      VARD4(I,J)=A*(VAD(I,J)-269.+(VAM(I,J)-59.)/60.+(VAS(I,J)-60.)     C    002:01B9:1
     1/3600.)/4.                                                        C    002:01C4:4
      GO TO 320                                                         C    002:01C6:2
  318 VARD4(I,J)=A*(89.-VAD(I,J)+(59.-VAM(I,J))/60.+(60.-VAS(I,J)       C    002:01C6:5
     1)/3600.)/4.                                                       C    002:01D2:0
  320 CONTINUE                                                          C    002:01D3:5
      VAAVR(I)=VARD4(I,1)+VARD4(I,2)+VARD4(I,3)+VARD4(I,4)              C    002:01D6:0
      VADAVF(I)=VAAVR(I)/A                                              C    002:01DC:2
      NVADAV(I)=VADAVF(I)                                               C    002:01DF:1
      VADAVW(I)=NVADAV(I)                                               C    002:01E1:4
      VAMAVF(I)=(VADAVF(I)-VADAVW(I))*60.                               C    002:01E4:0
      NVAMAV(I)=VAMAVF(I)                                               C    002:01E6:0
      VAMAVW(I)=NVAMAV(I)                                               C    002:01EA:3
  330 VASAVF(I)=(VAMAVF(I)-VAMAVW(I))*60.                               C    002:01EC:5
  335 CONTINUE                                                          C    002:01F0:5
      WRITE(6,340)                                                      C    002:01F3:0
  340 FORMAT('0',/,/,37X,"SUMMARY OF AVERAGE VALUES OF INPUT DATA FROM  C    002:01F7:2
     /THIS SURVEY")                                                     C    002:01F7:2
      WRITE(6,350)                                                      C    002:01F7:2
  350 FORMAT('0','STATION',23X,'AVERAGE BEARING DIFFERENCE',23X,'AVERAGE  C    002:01FB:2
     / VERTICAL',24X,'AVERAGE SLOPE')                                   C    002:01FB:2
      WRITE(6,360)                                                      C    002:01FB:2
  360 FORMAT(' ','NUMBER',24X,'FROM FIXED BEARING STATION',28X,         C    002:01FF:2
     /'ANGLE',32X,'DISTANCE')                                           C    002:01FF:2
      WRITE(6,370)                                                      C    002:01FF:2
  370 FORMAT(' ',34X,'DEGS',3X,'MINS',3X,'SECS',26X,'DEGS',             C    002:0203:2
     /3X,'MINS',3X,'SECS')                                              C    002:0203:2
      DO 386 I=1,N                                                      C    002:0203:2
      IF(I.EQ.NSET)GO TO 382                                            C    002:0204:0
      WRITE(6,380)I,NAVDD(I),NAVDM(I),AVDSF(I),NVADAV(I),NVAMAV(I),     C    002:0205:2
     /VASAVF(I),SDIST(I)                                                C    002:0214:2
  380 FORMAT(' ',2X,I2,30X,I4,3X,I3,4X,F4.0,26X,I3,4X,I3,4X,F4.0,25X,   C    002:021B:2
     1F8.3)                                                             C    002:021B:2
      GO TO 386                                                         C    002:021B:2
  382 WRITE(6,384)I                                                     C    002:021B:5
  384 FORMAT(3X,I2,32X,'NOT APPLICABLE',30X,'NOT APPLICABLE',24X,       C    002:0222:2
     1'NOT APPLICABLE')                                                 C    002:0222:2
  386 CONTINUE                                                          C    002:0222:2
C                                                                       C    002:0224:3
C     CALCULATION OF THE RESULTS OF THIS SURVEY                         C    002:0224:3
  390 FORMAT(5F5.3)                                                     C    002:0224:3
      DO 410 I=1,N                                                      C    002:0224:3
      C2(I)=1.+TAN(VAAVR(I))*TAN(VAAVR(I))                              C    002:0226:0
      IF(I.EQ.NFIX) GO TO 395                                           C    002:022B:1
      IF(I.EQ.NSET) GO TO 400                                           C    002:022C:3
      C1=HT-HTT+HDT-HD                                                  C    002:022D:5
      FSQRT(I)=SQRT(4.*SDIST(I)*SDIST(I)*C2(I)-4.*C1*C1)                C    002:0230:1
      HORDIS(I)=(-2.*C1*TAN(VAAVR(I))+FSQRT(I))/(2.*C2(I))              C    002:0237:3
      VERDIS(I)=HORDIS(I)*TAN(VAAVR(I))+HT-HTT                          C    002:023E:3
      GO TO 410                                                         C    002:0243:4
  395 C1=HT-HTST+HDT-HD                                                 C    002:0244:1
      FSQRT(I)=SQRT(4.*SDIST(I)*SDIST(I)*C2(I)-4.*C1*C1)                C    002:0246:3
      HORDIS(I)=(-2.*C1*TAN(VAAVR(I))+FSQRT(I))/(2.*C2(I))              C    002:024D:5
      VERDIS(I)=HORDIS(I)*TAN(VAAVR(I))+HT-HTST                         C    002:0254:5
      GO TO 410                                                         C    002:025A:0
  400 HORDIS(I)=0.                                                      C    002:025A:3
      VERDIS(I)=0.                                                      C    002:025C:0
  410 CONTINUE                                                          C    002:025D:3
C     ACTUAL BEARING(I)=FIXED BEARING-BEARING DIFFERENCE(I)             C    002:025F:4
      BEARFR=A*(BEARDI(NFIX)+BEARMI(NFIX)/60.+BEARSI(NFIX)/3600.)       C    002:025F:4
      DO 420 I=1,N                                                      C    002:0265:1
      IF(I.EQ.NSET)GO TO 420                                            C    002:0266:0
      ACBEAR(I)=BEARFR-AVDIFF(I)                                        C    002:0267:2
      BEARDF(I)=ACBEAR(I)/A                                             C    002:026A:1
      NBEARD(I)=BEARDF(I)                                               C    002:026D:0
      BEARDW(I)=NBEARD(I)                                               C    002:026F:3
      BEARMF(I)=(BEARDF(I)-BEARDW(I))*60.                               C    002:0271:5
      NBEARM(I)=BEARMF(I)                                               C    002:0275:5
      BEARMW(I)=NBEARM(I)                                               C    002:0278:2
      BEARSF(I)=(BEARMF(I)-BEARMW(I))*60.                               C    002:027A:4
  420 CONTINUE                                                          C    002:027E:4
      DO 430 I=1,N                                                      C    002:0280:5
      COORFN(I)=COORIN(NSET)+HORDIS(I)*COS(ACBEAR(I))                   C    002:0282:0
```

```
      COORFE(I)=COORIE(NSET)+HORDIS(I)*SIN(ACBEAR(I))                   C  002:0287:2
  430 RLF(I)=RLI(NSET)+VERDIS(I)                                        C  002:028C:4
      WRITE(6,440)                                                      C  002:0292:2
  440 FORMAT('0',/,/,48X,"SUMMARY OF RESULTS FROM THIS SURVEY")         C  002:0296:2
      WRITE(6,160)                                                      C  002:0296:2
      WRITE(6,190)                                                      C  002:029A:2
      DO 458 I=1,N                                                      C  002:029E:2
      IF(I.EQ.NSET)GO TO 452                                            C  002:029F:0
      WRITE(6,450)I,COORFN(I),COORFE(I),RLF(I),NBEARD(I),NBEARM(I),     C  002:02A0:2
     /BEARSF(I),HORDIS(I),VERDIS(I)                                     C  002:02AF:2
  450 FORMAT(' ',2X,I2,13X,F10.3,12X,F8.3,12X,F7.3,11X,F4.0,2X,F4.0,2X, C  002:02B8:2
     ?F4.0,12X,F8.3,11X,F7.3)                                           C  002:02B8:2
      GO TO 458                                                         C  002:02B8:2
  452 WRITE(6,455)I,COORFN(I),COORFE(I),RLF(I)                          C  002:02B8:5
  455 FORMAT(' ',2X,I2,13X,F10.3,12X,F8.3,12X,F7.3)                     C  002:02C6:2
  458 CONTINUE                                                          C  002:02C6:2
C                                                                       C  002:02C6:3
C     COMPARE INITIAL SURVEY WITH THIS SURVEY                           C  002:02C6:3
      DO 530 I=1,N                                                      C  002:02C6:3
      DELTAN(I)=COORFN(I)-COORIN(I)                                     C  002:02CA:0
      DELTAE(I)=COORFE(I)-COORIE(I)                                     C  002:02CD:3
      DELTAR(I)=RLF(I)-RLI(I)                                           C  002:02D1:0
      DELTAT(I)=SQRT(DELTAN(I)*DELTAN(I)+DELTAE(I)*DELTAE(I)+DELTAR(I)  C  002:02D4:3
     1*DELTAR(I))                                                       C  002:02DB:4
      RATEMM(I)=DELTAT(I)/TSIRIM                                        C  002:02DD:2
      IF(DELTAN(I).EQ.0.)GO TO 480                                      C  002:02E0:1
      BMID(I)=(ATAN(DELTAE(I)/DELTAN(I)))/A                             C  002:02E2:0
      IF(DELTAN(I).GT.0.)GO TO 460                                      C  002:02E6:4
      BMID(I)=180.+BMID(I)                                              C  002:02E8:3
      GO TO 510                                                         C  002:02EB:2
  460 IF(DELTAE(I)-0.)470,510,510                                       C  002:02EB:5
  470 BMID(I)=360.+BMID(I)                                              C  002:02ED:4
      GO TO 510                                                         C  002:02F0:4
  480 IF(DELTAE(I)-0.)490,500,500                                       C  002:02F1:1
  490 BMID(I)=270.                                                      C  002:02F3:0
      GO TO 510                                                         C  002:02F4:5
  500 BMID(I)=90.                                                       C  002:02F5:2
  510 IF(DELTAT(I).EQ.0.)GO TO 520                                      C  002:02F7:0
      ANGELM(I)=(ARSIN(DELTAR(I)/DELTAT(I)))/A                          C  002:02F8:5
      GO TO 530                                                         C  002:02FD:3
  520 ANGELM(I)=90.                                                     C  002:02FE:0
  530 CONTINUE                                                          C  002:02FF:4
      WRITE(6,540)                                                      C  002:0301:5
  540 FORMAT('0',/,/,56X,"CALCULATED MOVEMENTS")                        C  002:0306:2
      WRITE(6,550)                                                      C  002:0306:2
  550 FORMAT('0','STATION',8X,'CHANGE IN COORDINATES',9X,'CHANGE IN',   C  002:030A:2
     19X,'MAGNITUDE OF',8X,'MOVEMENT',9X,'DIRECTION OF MOVEMENT IN DEGRE C  002:030A:2
     /ES')                                                              C  002:030A:2
      WRITE(6,560)                                                      C  002:030A:2
  560 FORMAT(' ','NUMBER',11X,'NORTH',7X,'EAST',10X,'REDUCED LEVEL',9X, C  002:030E:2
     ?'MOVEMENT',9X,'PER MONTH',12X,'BEARING',8X,'ELEVATION')           C  002:030E:2
      DO 568 I=1,N                                                      C  002:030E:2
      IF(I.EQ.NSET) GO TO 566                                           C  002:030F:0
  565 WRITE(6,570)I,DELTAN(I),DELTAE(I),DELTAR(I),DELTAT(I),            C  002:0310:2
     ?RATEMM(I),BMID(I),ANGELM(I)                                       C  002:0310:1
      GO TO 568                                                         C  002:0326:2
  566 WRITE(6,567)I,DELTAN(I),DELTAE(I),DELTAR(I),DELTAT(I),RATEMM(I)   C  002:0326:5
  567 FORMAT(2X,I2,13X,F6.3,5X,F6.3,12X,F6.3,14X,F6.3,11X,F8.5)         C  002:0338:2
  570 FORMAT(2X,I2,13X,F6.3,5X,F6.3,12X,F6.3,14X,F6.3,11X,F8.5,13X,F7.3, C  002:0338:2
     ?9X,F7.3)                                                          C  002:0338:2
  568 CONTINUE                                                          C  002:0338:2
      WRITE(6,580)                                                      C  002:033A:3
  580 FORMAT('0',45X,'**********CALCULATIONS COMPLETE**********')       C  002:033E:2
      STOP                                                              C  002:033E:2
      END                                                               C  002:033F:1
                                                               SEGMENT 002 IS 037B LONG
```

7.2 APPENDIX 2 - HAWKSWOOD CUT DRILL CORE LOGS.

LOG OF DRILL HOLE

HOLE NO. one

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 29.31m
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)
Top soil Flight augers 0-3.04m.	28 27 26	
Light yellowish brown-light grayish brown, mottled rusty brown, faintly fissured, slightly weathered, clayey SILT, with fissures moderately-steeply inclined, and weathering along fissures.	25 24 23	Light yellowish brown-light grayish brown, mottled rusty brown, clayey SILT, stiff, moist-wet, slightly plastic CL ? Unit 2, Bed B
Light bluish gray, faintly bedded, clayey SILT, with bedding laminated-very thinly bedded, sub horizontal	22 21 20 19	Light bluish gray, clayey SILT, firm -stiff, moist-wet, slightly plastic, with occasional very thinly interbedded sand lenses ML ? Unit 2, Bed C

Other column headings: WEATHERING (SW MW HW), HARDNESS (H MH MS S), POINT LOAD TEST (kPa), CORE LOSS/LIFT %, Core size casing, GRAPHIC LOG, FRACTURE LOG (Spacing of natural fractures, cms), DATE/DEPTH, R.Q.D., WATER LEVEL, DRILL WATER LOSS % (0-100).

DRILLER: McVICAR
STARTED:
FINISHED:
DRILL:

WEATHERING: UW - Unweathered; SW - Slightly weathered; MW - Moderately weathered; HW - Highly weathered; CW - Completely weathered

HARDNESS: VH - Very hard; H - Hard; MH - Moderately hard; MS - Moderately soft; S - Soft; VS - Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: G. GROCOTT
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 1 OF 3

LOG OF DRILL HOLE

HOLE NO. one

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 29.31

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (Formation name; Rock or soil type; Description of core (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	Depth H.A.D.	ROCK STRUCTURES (Defects) / (OR SOIL DESCRIPTION)
	18, 17	? Unit 2, Bed C
Light brown, well bedded, slightly weathered, clayey SILT, with some sand, with bedding laminated -very thinly bedded, sub horizontal	16, 15	Light brown, clayey SILT, moist, stiff, non plastic, with very thinly interbedded lenses of clayey silt and sand CL ? Unit 3, Bed H
Light bluish gray, faintly bedded, clayey SILT, with some sand, with bedding laminated-very-thinly bedded, sub horizontal	14, 13, 12, 10, 9, 8	Light bluish gray, clayey Silt, with some sand, firm-stiff, moist-wet, slightly plastic, with rare very thinly interbedded lenses of clayey silt and sand ML ? Unit 4, Bed J

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING: UW - Unweathered; SW - Slightly weathered; MW - Moderately weathered; HW - Highly weathered; CW - Completely weathered

HARDNESS: VH - Very hard; H - Hard; MH - Moderately hard; MS - Moderately soft; S - Soft; VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED

DATE

TRACED

CHECKED

VERTICAL SCALE

SHEET 2 OF 3

HOLE NO. one

LOG OF DRILL HOLE

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 29.31m.
ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour)	SW MW HW WEATHERING	VH H MH MS S HARDNESS	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D.	Core size casing	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS % 0-100
									? Unit 4, Bed J				
					7								
Notes: 1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km. 2. Three PVC pipes slotted at: 3.4-5.3m 8.2-10.5m 18.5-20.4m													

DRILLER: ....
STARTED: ....
FINISHED: ....
DRILL: ....

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: ....
DATE: ....
TRACED: ....
CHECKED: ....
VERTICAL SCALE: ....

SHEET 3 OF 3

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 36.48m
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D (Core size casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS %
Top soil Flight augers 0-3.04m. No recovery					35 34 33							
No core					32 31							
Light grayish brown, mottled rusty brown, faintly bedded, occasional fissures, slightly weathered, clayey SILT and SAND, with bedding laminated-very thinly bedded, sub horizontal with fissures steeply inclined					30 29 28 27 26			Light grayish brown, mottled rusty brown, clayey SILT and SAND, soft-firm, moist-wet, slightly plastic, with occasional very thinly interbedded silt and sand lenses ML-SM ? Unit 2, Bed B				

DRILLER: McVicAR
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms)
500 100 50 10 5 1
Spacing of natural fractures
1 2 10 20 100 >100
Fractures/m of core

EXPLANATION

LOGGED GROCOTT
DATE
TRACED
CHECKED
VERTICAL SCALE
SHEET 1 OF 3

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 36.48m
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME, ROCK OR SOIL TYPE, DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)
	25	
	24	
	23	
Light bluish gray, faintly bedded, rare fissures, slightly weathered, clayey sandy SILT, with bedding laminated-very thinly bedded, sub horizontal, some fissures clay coated, with fissures steeply inclined.	22	Light bluish gray, clayey sandy SILT, firm, moist, slightly plastic, with very thinly bedded sub horizontal sand lenses ML-SM ? Unit 2, Bed C ? Unit 4, Bed J
	21	
	20	
	19	
	18	
	17	
	16	

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED
DATE
TRACED
CHECKED
VERTICAL SCALE

SHEET 2 OF 3

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 36.48
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME, ROCK OR SOIL TYPE, DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD (Core size, casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) cms	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	RQD	WATER LEVEL Date	DRILL WATER LOSS % 0-100
Light bluish gray, faintly bedded, rare fissures, slightly weathered, clayey sandy SILT, with bedding laminated-very thinly bedded, sub horizontal, some fissures clay coated, with fissures steeply inclined.					15, 14, 13, 12, 11, 10, 9			Light bluish gray, clayey sandy SILT, firm, moist, slightly plastic, with very thinly bedded sub horizontal sand lenses ML-SM ? Unit 2, Bed C ? Unit 4, Bed J				

Notes:
1. All R.L.s relative to culvert invert of R.L. 16.19m. at140.7km.
2. Two PVC pipes slotted at: 4.6-6.4m
25.2-27.4m

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core
100 50 10 5 1 — 1 2 10 20 100 1000

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 3 OF 3

LOG OF DRILL HOLE	HOLE NO. three

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.02

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour)	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D (Core size casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS % 0–100
Top soil Flight augers 0-2.9m. No sample recovery					38 37							
Light brown, rusty brown mottled, faintly bedded, fissured, slightly weathered, clayey SILT, with some sand, with bedding laminated, sub horizontal, with fissures steeply inclined and slightly weathered					36 35 34 33 32 31 30 29			Light brown, mottled rusty brown, clayey SILT with some sand, firm, moist, slightly plastic, with very thinly bedded, sub horizontal sand lenses ML-SM ? Unit 2, Bed B				

DRILLER: McVICAR

STARTED:

FINISHED:

DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: GROSOTT
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:

LOG OF DRILL HOLE

HOLE NO. three

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.02m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) (OR SOIL DESCRIPTION)
Light bluish gray, faintly bedded, slightly weathered, clayey SILT, with bedding laminated, sub horizontal	28, 27, 26, 25	Light bluish gray, clayey SILT, firm-stiff, moist-wet, slightly plastic, with very thinly bedded rusty brown lenses ML ? Unit 2, Bed C
Light bluish gray, homogeneous, slightly weathered, clayey SILT	24, 23, 22, 21, 20, 19	Light bluish gray, slightly weathered, clayey SILT, firm -stiff, moist-wet, slightly plastic, ML ? Unit 4, Bed J

Column headings: WEATHERING (SW MW HW); HARDNESS (H MH MS S); POINT LOAD TEST (kPa); CORE LOSS/LIFT %; Core size, casing; GRAPHIC LOG; FRACTURE LOG (Spacing of natural fractures) (cms); DATE/DEPTH; R.Q.D.; WATER LEVEL (Date); DRILL WATER LOSS % (0–100)

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED:

DATE:

TRACED:

CHECKED:

VERTICAL SCALE:

SHEET 2 OF 3

LOG OF DRILL HOLE

HOLE NO. three

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.02
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD (Core size, casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS %
Light bluish gray, homogeneous, slightly weathered, clayey SILT					18 17 16 15			Light bluish gray, slightly weathered, clayey SILT, firm -stiff, moist-wet, slightly plastic, ML ? Unit 4, Bed J				

Notes:
1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km.
2. Two PVC pipes slotted at: 9,5-11.3m
22.2-24.4m

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG
(cms) 100 50 10 5 1 0
Spacing of natural fractures
1 2 10 20 100 1000
Fractures/m of core

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 3 OF 3

HOLE NO.	four

LOG OF DRILL HOLE

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.0

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD (Core size casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	RQD	WATER LEVEL (Date)	DRILL WATER LOSS % (0-100)
Top soil Brown CLAY and stones Brown CLAY Flight auger 0-3.05m Blue SILT, very soft								(Fill)				
Light bluish gray, light-dark brown, clayey sandy SILT, with some sub angular gravels (Fill)								Light bluish gray, light-dark brown, clayey sandy SILT, with some sub angular gravels, soft, moist-wet, slightly plastic				
Light brown, mottled light gray, homogeneous slightly weathered, clayey SILT, with some rounded-sub angular, rough gravels								Light brown, mottled light gray, clayey SILT, with some rounded-sub angular gravels, firm, moist, slightly plastic CL ? Unit 2, Bed B				

DRILLER: McVICAR

STARTED:

FINISHED:

DRILL:

WEATHERING: UW - Unweathered; SW - Slightly weathered; MW - Moderately weathered; HW - Highly weathered; CW - Completely weathered

HARDNESS: VH - Very hard; H - Hard; MH - Moderately hard; MS - Moderately soft; S - Soft; VS - Very soft

FRACTURE LOG (cms): 100 50 10 5 1 — Spacing of natural fractures; 1 2 10 20 100 — Fractures/m of core

EXPLANATION

LOGGED: GROCOTT

DATE:

TRACED:

CHECKED:

VERTICAL SCALE:

SHEET 1 OF 3

LOG OF DRILL HOLE

HOLE NO. four

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM H.A.D. GROUND 39.01
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME. ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	SW MW HW WEATHERING	H MH MS S HARDNESS	POINT LOAD TEST (kPa)	CORE LOSS/LIFT % 5 10 50	DEPTH HAD Core size casing m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) cms 50 10 5	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE, DEPTH	R.Q.D. %	WATER LEVEL Date	DRILL WATER LOSS % 0-100
Light bluish gray-light greenish gray, faintly bedded, occasionally fissured, slightly weathered, clayey sandy SILT, with bedding laminated, sub horizontal, with fissures sub horizontal -steeply inclined								Light bluish gray, light greenish gray, clayey sandy SILT, soft-firm, moist, slightly plastic, with very thinly bedded, rusty brown, sub horizontal sand lenses ML ? Unit 2, Bed C				
Light greenish gray-dark brown, well bedded, sandy CLAY and SILT, with some light gray, loose, sub angular gravels, with bedding consisting of very thinly layered sub horizontal carbonaceous bands ? Unit 4, Bed I, with possibly gravels from Unit 3, Bed E								Light greenish gray dark brown, sandy CLAY and SILT, firm, moist, slightly -moderately plastic, with some light gray, loose, sub angular gravels, with layering consisting of very thinly bedded, sub horizontal carbonaceous bands OH-ML				

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) 100 50 10 5 1 — Spacing of natural fractures; 1 2 10 20 100 — Fractures/m of core

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 2 OF 3

LOG OF DRILL HOLE

HOLE NO. four

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.01

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour).	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD Core size casing	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	RQD	WATER LEVEL Date	DRILL WATER LOSS % 0-100
								? Unit 4, Bed I, with possibly gravels from Unit 3, Bed E				

Notes:
1. All R.L.s relative to culvert invert of R.L. 16.19m. at 140.7km.
2. Three PVC pipes slotted at: 4.6-6.6m
10.6-12.6m
18.9-21.3m

DRILLER:	WEATHERING	HARDNESS	FRACTURE LOG (cms)	
STARTED:	UW - Unweathered SW - Slightly weathered MW - Moderately weathered HW - Highly weathered CW - Completely weathered	VH - Very hard H - Hard MH - Moderately hard MS - Moderately soft S - Soft VS - Very soft	100 50 10 5 1 0.1 Spacing of natural fractures 1 2 10 20 100 1000 Fractures/m of core	LOGGED DATE TRACED CHECKED VERTICAL SCALE
FINISHED:	EXPLANATION			
DRILL:				SHEET 3 OF 3

LOG OF DRILL HOLE

HOLE NO. five

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 34.5

ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE (FORMATION NAME: / ROCK OR SOIL TYPE: / DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) (OR SOIL DESCRIPTION)
Flight augers 0-2.9m No recovery	33.5 32.5	
Light yellowish brown, faintly bedded, fissured, slightly weathered, clayey SILT, with some sand, with bedding laminated -very thinly bedded, sub horizontal, bedding increasing towards bottom of unit, with weathering along fissure faces as well as slight mottling	31.5 30.5 29.5 28.5 27.5 26.5 25.5 24.5	Light yellowish brown, mottled light brown, clayey SILT, with some sand, firm-stiff, moist, non plastic -slightly plastic, with very thinly interbedded sand and silt lenses towards bottom of unit ML-SM ? Unit 2, Bed B

Other column headings: WEATHERING (SW MW HW), HARDNESS (H MH MS S), POINT LOAD TEST (kPa), CORE LOSS/LIFT %, Core size, casing, GRAPHIC LOG, FRACTURE LOG (Spacing of natural fractures) cms, JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness), (consistency, compactness, water content, group symbol etc.), DATE/DEPTH, RQD, WATER LEVEL Date, DRILL WATER LOSS % 0-100

DRILLER: McVicar

STARTED:

FINISHED:

DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED GROCOTT

DATE ....

TRACED ....

CHECKED ....

VERTICAL SCALE ....

SHEET 1 OF 3

LOG OF DRILL HOLE

HOLE NO. Five

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 34.51
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D. (m)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS, SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL Date	DRILL WATER LOSS % 0–100
Light bluish gray, very faintly bedded, occasional fissures, slightly weathered, clayey sandy SILT, with bedding laminated, sub horizontal, with weathering along fissures (Chlamys delicatula at 14.9m)					23.5 22.5 21.5 20.5 19.5 18.5 17.5 16.5 15.5 14.5			Light bluish gray, clayey sandy SILT, firm stiff, moist, non plastic, with very thinly inter bedded, sub horizontal light gray and brown sand lenses ML-SM ? Unit 2, Bed C ? Unit 4, Bed J.				

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms)
Spacing of natural fractures
Fractures/m of core

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:

SHEET 2 OF 3

HOLE NO. Five

LOG OF DRILL HOLE

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 34.51

ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE (FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (VH H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD (Core size casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing smoothness) (OR SOIL DESCRIPTION) (consistency compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL Date	DRILL WATER LOSS % 0–100
Light bluish gray, very faintly bedded, occasional fissures, slightly weathered, clayey sandy SILT, with bedding laminated -sub horizontal, with weathering along fissures					13.5 12.5			Light bluish gray, clayey sandy SILT, firm-stiff, moist, non plastic, with very thinly inter bedded, sub horizontal light gray and brown sand lenses. ML-SM ? Unit 2, Bed C ? Unit 4, Bed J				

Notes:
1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km.
2. Slotted PVC pipe to 22.2m.

DRILLER: ....

STARTED: ....

FINISHED: ....

DRILL: ....

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG (cms)
Spacing of natural fractures
Fractures/m of core

EXPLANATION

LOGGED: ....
DATE: ....
TRACED: ....
CHECKED: ....
VERTICAL SCALE: ....

SHEET 3 OF 3

LOG OF DRILL HOLE

HOLE NO. six

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 38.42m
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) / (OR SOIL DESCRIPTION)
Light gray-light brown, finer grained material occasionally fissured, slightly weathered, clayey sandy SILT and GRAVELS and COBBLES, without any visible bedding ? Unit 2, Bed B	27.4, 26.4, 25.4	Light brown, rare fissures, slightly weathered, clayey SILT and GRAVELS, soft-firm, loose, moist-wet, fines slightly plastic, with gravels rounded-sub angular rough ML-GP
	24.4, 23.4, 22.4, 21.4	Light gray, slightly weathered, clayey silty GRAVELS and COBBLES with particles loose, dry, rounded smooth GM (cuttings only)
Light brown, grading downwards through greenish gray to bluish gray, fissured, slightly weathered, clayey sandy SILT ? Unit 2, Bed C	20.4, 19.4	Light brown, green -ish gray-bluish gray, fissured, slightly weathered, clayey sandy SILT, Moist, firm, slightly plastic.
	18.4	

DRILLER: mcvicar
STARTED:
FINISHED:
DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED G.R.O.S.O.T.T.
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 1 OF 3

LOG OF DRILL HOLE

HOLE NO. six

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 38.42m

ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE:(grain size, texture, mineral content, hardness, strength, cement & matrix colour).	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT % 5 10 50	DEPTH HAD Core size, casing m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) 50 10 5 cms	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES; FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE DEPTH	RQD	WATER LEVEL Date	DRILL WATER LOSS % 0–100
Top soil-some stones Flight augers 0-3.04m Brownish gray CLAY, very wet					37.4 36.4 35.4			(Fill)				
Light grayish brown, mottled creamey brown, slightly weathered, homogeneous, clayey sandy SILT and GRAVELS					34.4			Light grayish brown, mottled creamey brown, clayey sandy SILT and GRAVELS, soft-firm, moist, non plastic (Fill)				
Light gray-light brown, homogeneous, fresh-slightly weathered, GRAVELS					33.4 32.4 31.4 30.4			Light gray-light brown, GRAVELS, loose, moist-wet, non plastic, with gravels rounded-sub angular, rough-smooth GP ? Unit 1, Bed A				
					29.4 28.4			Light brown-cream, mottled rusty brown, homogeneous, slightly weathered, clayey SILT, soft-firm, moist, slightly plastic				

DRILLER: ....

STARTED: ....

FINISHED: ....

DRILL: ....

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG
(cms) 100 50 10 5 1 — 2 10 20 100
Spacing of natural fractures
Fractures/m of core

EXPLANATION

LOGGED: ....
DATE ....
TRACED: ....
CHECKED: ....
VERTICAL SCALE ....

SHEET 2 OF 3

EG 3/25 M

LOG OF DRILL HOLE

HOLE NO. six

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 38.42m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME, ROCK OR SOIL TYPE; DESCRIPTION OF CORE: grain size, texture, mineral content, hardness, strength, cement & matrix colour)	DEPTH H.A.D	GRAPHIC LOG	ROCK STRUCTURES (Defects) (OR SOIL DESCRIPTION)
Light yellowish brown-light bluish gray-rusty brown, occasionally fissured, slightly-moderately weathered, sandy SILT GRAVELS and COBBLES, occasional lensing of gravels and finer grained material ? Unit 3, Bed E&F	17.4, 16.4, 15.4, 14.4		Light yellowish brown-light bluish gray, sandy SILT, with some gravels, firm, moist, non plastic, with very thinly bedded, sub horizontal silt and sand lenses, with gravels rounded-sub angular ML Light gray-rusty brown, matrix moderately weathered, GRAVELS, loose, dry-moist, with gravels rounded-sub angular and well cemented GP
Light bluish gray, well bedded, silty CLAY, with some gravels, with bedding very thinly bedded, sub horizontal	13.4, 12.4, 11.4, 10.4		Light bluish gray, Silty CLAY, with some gravels, firm, moist, slightly plastic, with gravels rounded-sub angular, with layering of finer grained material and gravels ML ? Unit 4, Bed J

Notes:
1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km.
2. PVC pipe to28.6m. Slotted from 27.4-28.6m.

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING: UW - Unweathered; SW - Slightly weathered; MW - Moderately weathered; HW - Highly weathered; CW - Completely weathered

HARDNESS: VH - Very hard; H - Hard; MH - Moderately hard; MS - Moderately soft; S - Soft; VS - Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED
DATE
TRACED
CHECKED
VERTICAL SCALE

SHEET 3 OF 3

LOG OF DRILL HOLE

HOLE NO. seven

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.38m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME: / ROCK OR SOIL TYPE: / DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D. (Core size, casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS % (0–100)
Top soil Flight augers 0–3.04m. Brown SILT with some gravels					38.4 37.4 36.4			(Fill)				
Light yellowish brown, homogeneous, clayey sandy SILT, with some gravels					35.4 34.4 33.4			Light yellowish brown, clayey sandy SILT, with some gravels, soft-firm, moist, non plastic-slightly plastic, with gravels sub angular-angular, rough ML ? Unit 2, Bed B				
Light bluish gray, homogeneous, fresh, clayey sandy SILT.					32.4			Light bluish gray, clayey sandy SILT, soft-firm, moist, non plastic-slightly plastic ML ? Unit 2, Bed C				
Light brown, faintly bedded, fresh, clayey sandy SILT, with some gravels, with very thinly bedded, sub horizontal gravel lenses					31.4 30.4 29.4			Light-dark brown, faintly bedded, fresh, clayey sandy SILT, soft-firm, moist-wet, slightly plastic ML No correlation poss.				

DRILLER: McVICAR

STARTED:

FINISHED:

DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: GROCOTT

DATE:

TRACED:

CHECKED:

VERTICAL SCALE:

SHEET 1 OF 3

HOLE NO.	seven

LOG OF DRILL HOLE

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 39.38
ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour)	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/ LIFT %	DEPTH HAD Core size, casing	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) cms	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL Date	DRILL WATER LOSS % 0–100
Light brown-light gray, fissured, slightly weathered, sandy SILT, with fissures steeply inclined, weathering mainly along fissures, no bedding visible					28.4 27.4 26.4 25.4 24.4 23.4 22.4 21.4 20.4 19.4			Light brown-light gray, sandy SILT, stiff, dry-moist, non plastic ML No correlation poss.				

DRILLER:	WEATHERING UW – Unweathered SW – Slightly weathered MW – Moderately weathered HW – Highly weathered CW – Completely weathered	HARDNESS VH – Very hard H – Hard MH – Moderately hard MS – Moderately soft S – Soft VS – Very soft	FRACTURE LOG (cms) 100 50 10 5 1 0 Spacing of natural fractures 1 2 10 20 100 1000 Fractures/m of core	LOGGED: DATE: TRACED: CHECKED: VERTICAL SCALE:
STARTED:				
FINISHED:	EXPLANATION			
DRILL:				SHEET 2 OF 3

LOG OF DRILL HOLE

HOLE NO. seven

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 39.38m
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content hardness strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D. (Core size casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY (attitude thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS % 0–100
					18.4	X X X X		Light brown-light gray, sandy SILT, stiff, dry-moist, non plastic ML No correlation poss.				
Notes: 1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km. 2. Slotted PVC to EoH.												

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG (cms) Spacing of natural fractures
Fractures/m of core

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:

SHEET 3 OF 3

LOG OF DRILL HOLE

HOLE NO. eight

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.
GRID REF. .... M.W.D. CO-ORD. .... DATUM Arbitary H.A.D. GROUND 38.64
ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE (FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT % (5 20 50)	DEPTH HAD (Core size casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) (50 10 5 cms)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness spacing, smoothness) (OR SOIL DESCRIPTION) (consistency compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL (Date)	DRILL WATER LOSS % (0-100)
Top soil Flight augers 0-3.04m (Fill) Brown silty CLAY					37.6 36.6 35.6							
Light brown-dark brownish gray, fissured, slightly weathered, sandy gravelly SILT, with fissures steeply inclined, with weathering along fissures					34.6 33.6			Light brown-dark brownish gray, sandy gravelly SILT, firm, moist, non plastic ML-GP (Fill) - gravel content increasing towards bottom of unit				
Light yellowish brown, homogeneous fresh, silty SAND, with some small gravels					32.6 31.6 30.6 29.6 28.6			Light yellowish brown, silty SAND, with some small gravels, soft-firm, moist-wet, non plastic, with gravels angular and rough ML-GP ? Unit 2, Bed B - light bluish gray silty SAND				

DRILLER: ....
STARTED: ....
FINISHED: ....
DRILL: ....

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG
(cms) 100 50 10 5 1 0.1 — Spacing of natural fractures
1 2 10 20 100 1000 — Fractures/m of core

EXPLANATION

LOGGED ....
DATE ....
TRACED ....
CHECKED ....
VERTICAL SCALE ....
SHEET ...... OF ......

LOG OF DRILL HOLE

HOLE NO. eight

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 38.64

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D.	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS, VEINS, SEAMS, SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS %
					27.6			ML-GP ? Unit 2, Bed B				
Light gray, homogeneous, fresh, GRAVELS, with few fines					26.6, 25.6, 24.6			Light gray, medium GRAVELS, clean, loose, moist, with gravels rounded-sub angular GP No correlation poss.				
Light brown-rusty brown-light bluish green, very faintly bedded, fresh, clayey silty SAND, with bedding due to only very gradational changes in grain size					23.6, 22.6			Light brown-rusty brown-light bluish green, clayey silty SAND, firm, moist, non plastic, with very weak sub horizontal bedding SM No correlation				
Light gray-light brown, homogeneous, slightly weathered, GRAVELS, clean					21.6, 20.6, 19.6, 18.6			Light gray-light brown, medium GRAVELS, loose, moist-wet, with few fines, with gravels sub angular -rounded, smooth-rough GP No correlation poss.				

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED:
DATE:
TRACED
CHECKED:
VERTICAL SCALE:
SHEET ... OF ...

LOG OF DRILL HOLE

HOLE NO. eight

PROJECT MSc Thesis FEATURE Hawkswood Cutting LOCATION N. Cant.

GRID REF. M.W.D. CO-ORD. DATUM Arbitary H.A.D. GROUND 38.64m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour)	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D. Core size casing	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE/DEPTH	R.Q.D.	WATER LEVEL Date	DRILL WATER LOSS % 0-100
						o o						
Notes: 1. All R.L.s relative to culvert invert of R.L. 16.19m at 140.7km. 2. Slotted PVC pipe to EoH.					17.6							

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED:

DATE:

TRACED

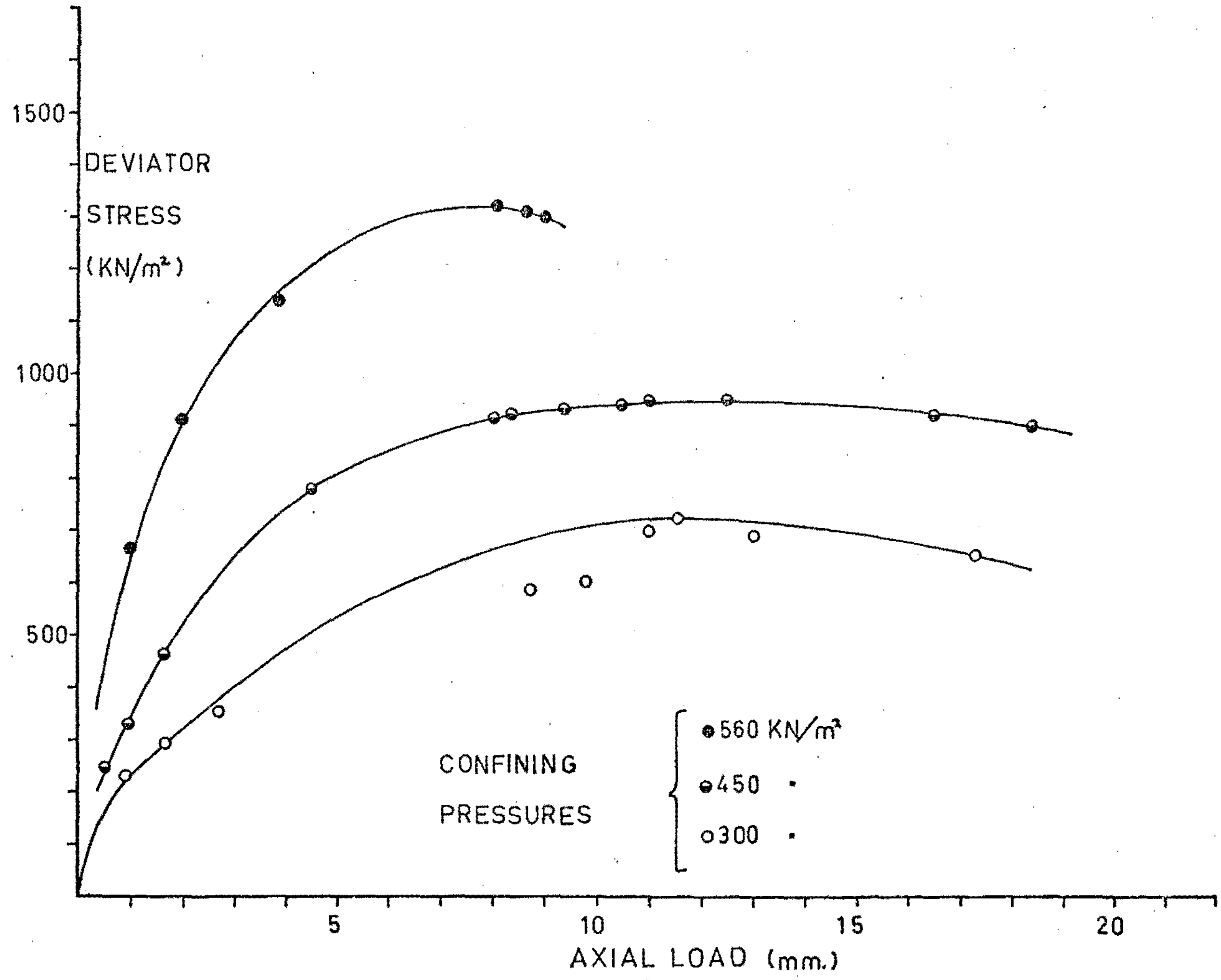
CHECKED:

VERTICAL SCALE

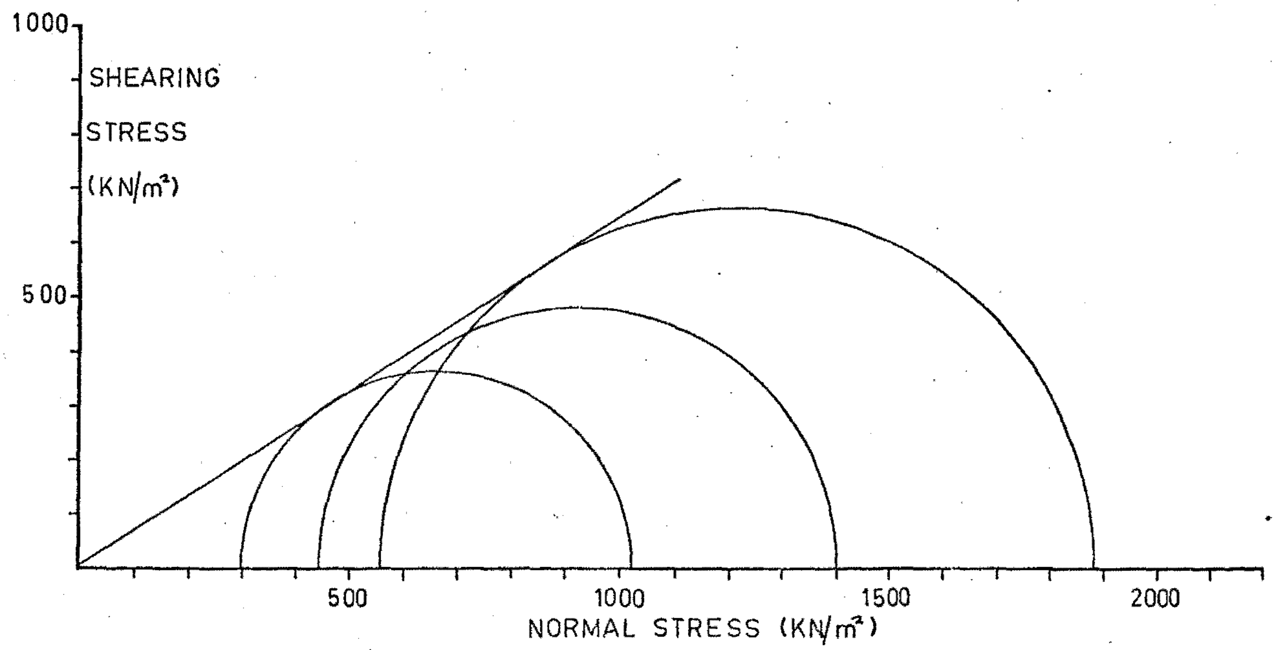
SHEET OF

7.3 APPENDIX 3 - MOHR'S FAILURE ENVELOPES.

UNIT 2, BED C

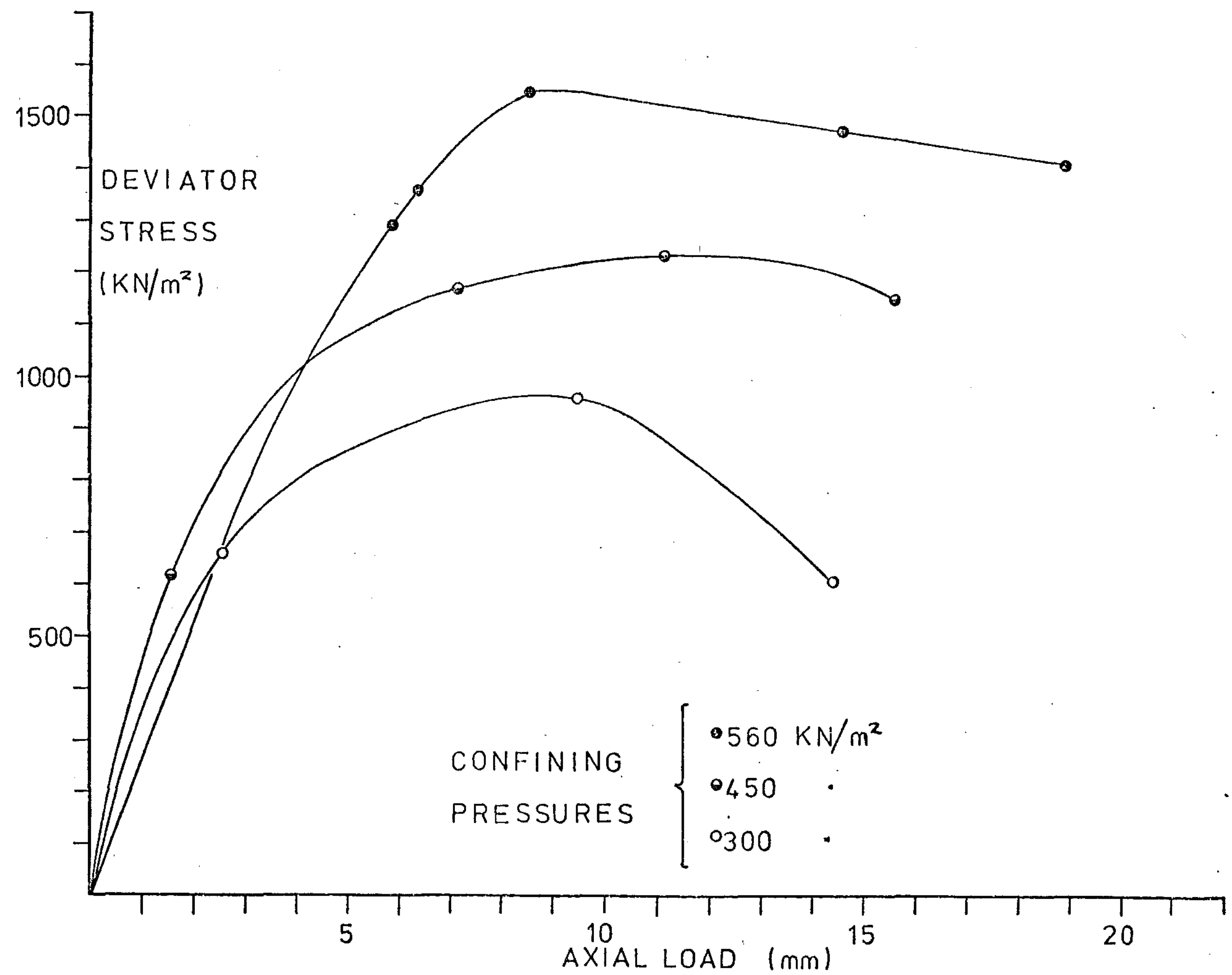


1500
1000
500
DEVIATOR
STRESS
(KN/m²)
5
10
15
20
AXIAL LOAD (mm.)
CONFINING
PRESSURES
560 KN/m²
450 "
300 "

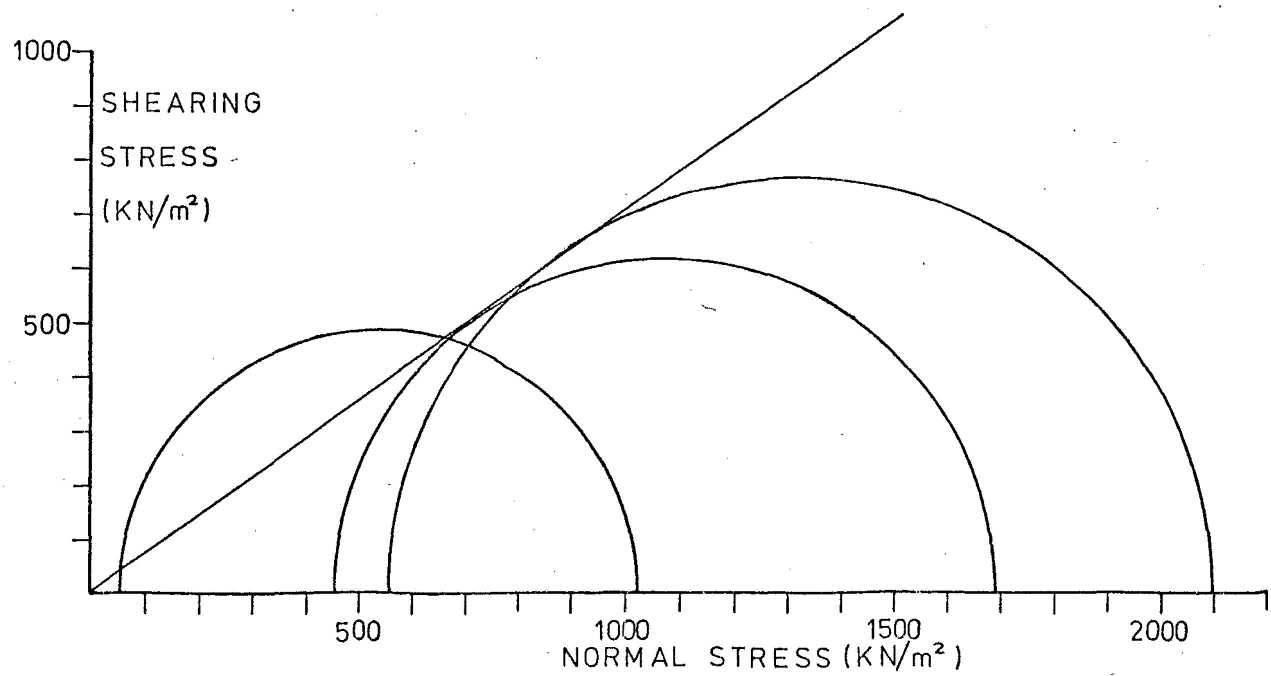


1000
500
SHEARING
STRESS
(KN/m²)
500
1000
1500
2000
NORMAL STRESS (KN/m²)

UNIT 3, BED G

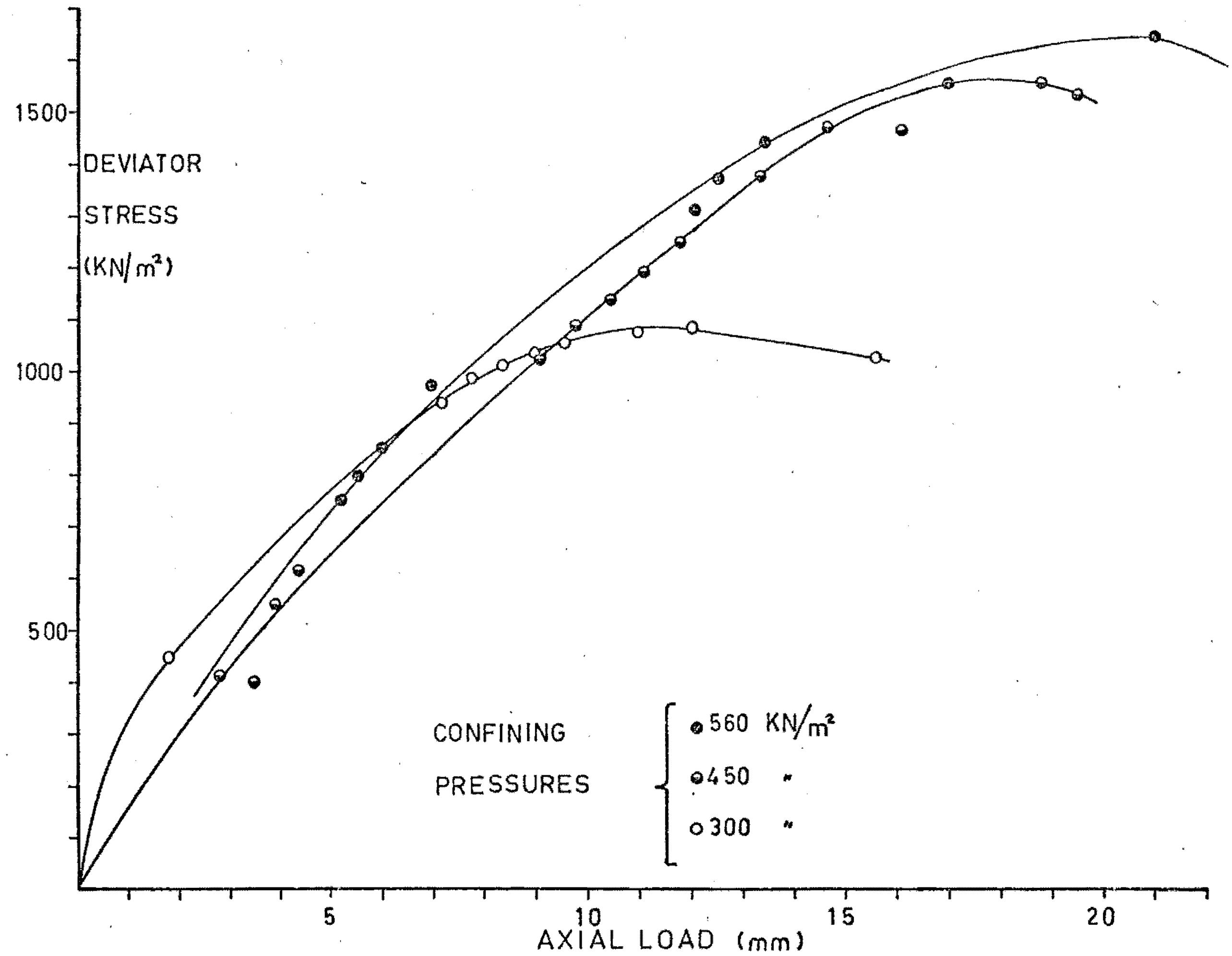


1500
1000
500
DEVIATOR
STRESS
(KN/m²)
CONFINING
PRESSURES
560 KN/m²
450
300
5
10
15
20
AXIAL LOAD (mm)

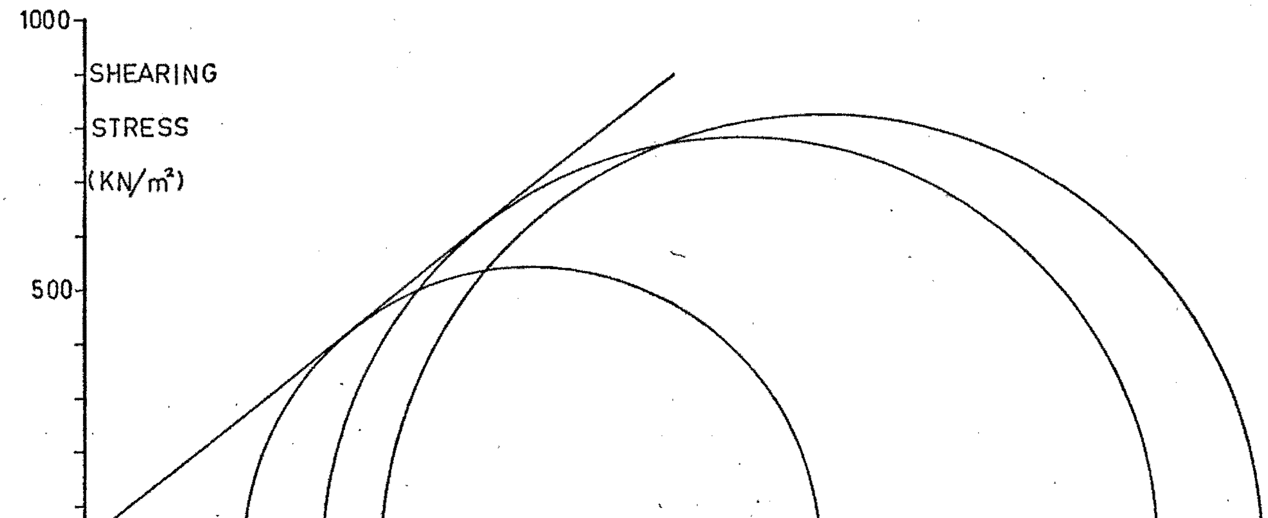


1000
500
SHEARING
STRESS
(KN/m²)
500
1000
1500
2000
NORMAL STRESS (KN/m²)

UNIT 4 , BED I

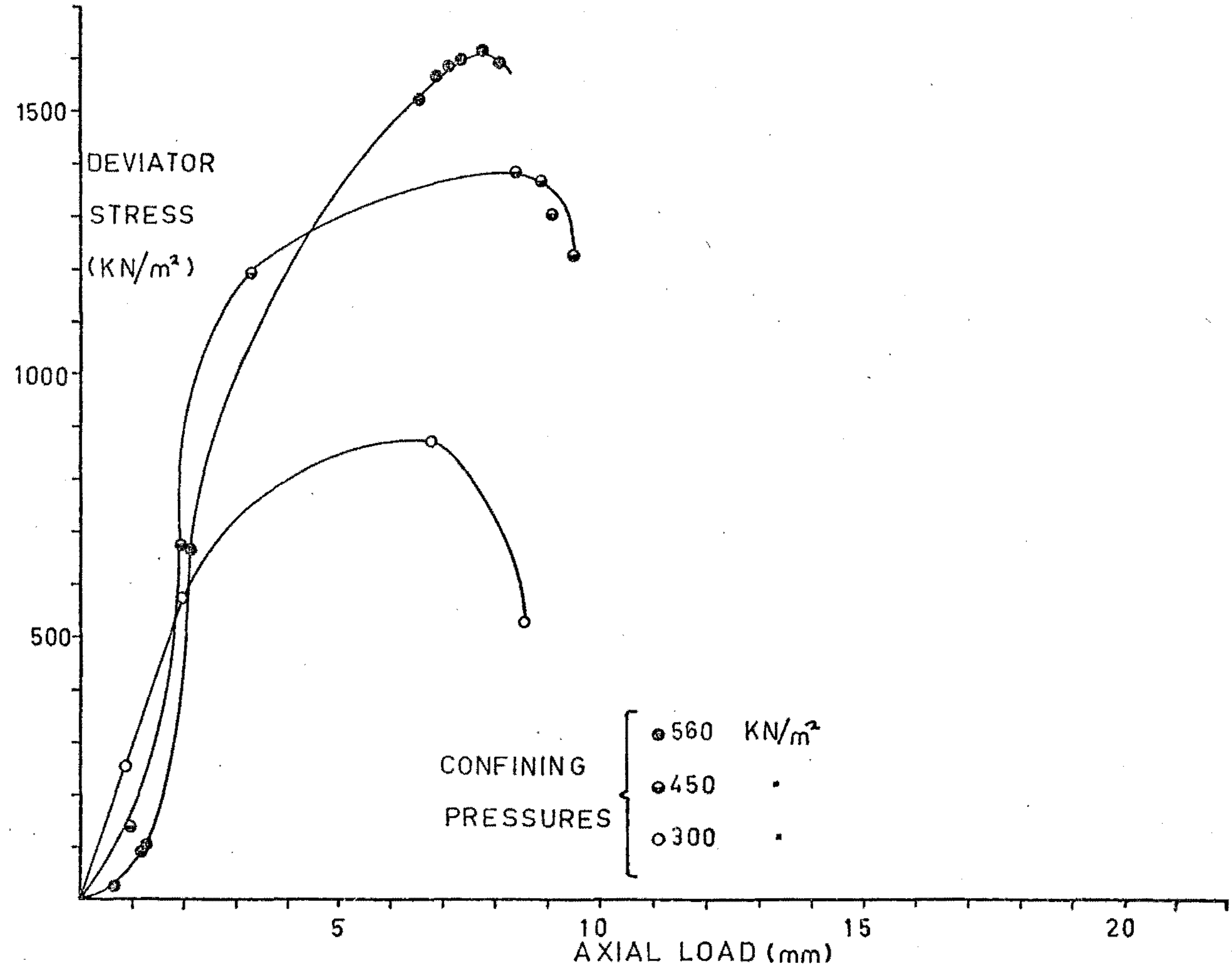


1500
DEVIATOR
STRESS
(KN/m²)
1000
500
CONFINING
PRESSURES
560 KN/m²
450 "
300 "
5
10
15
20
AXIAL LOAD (mm)

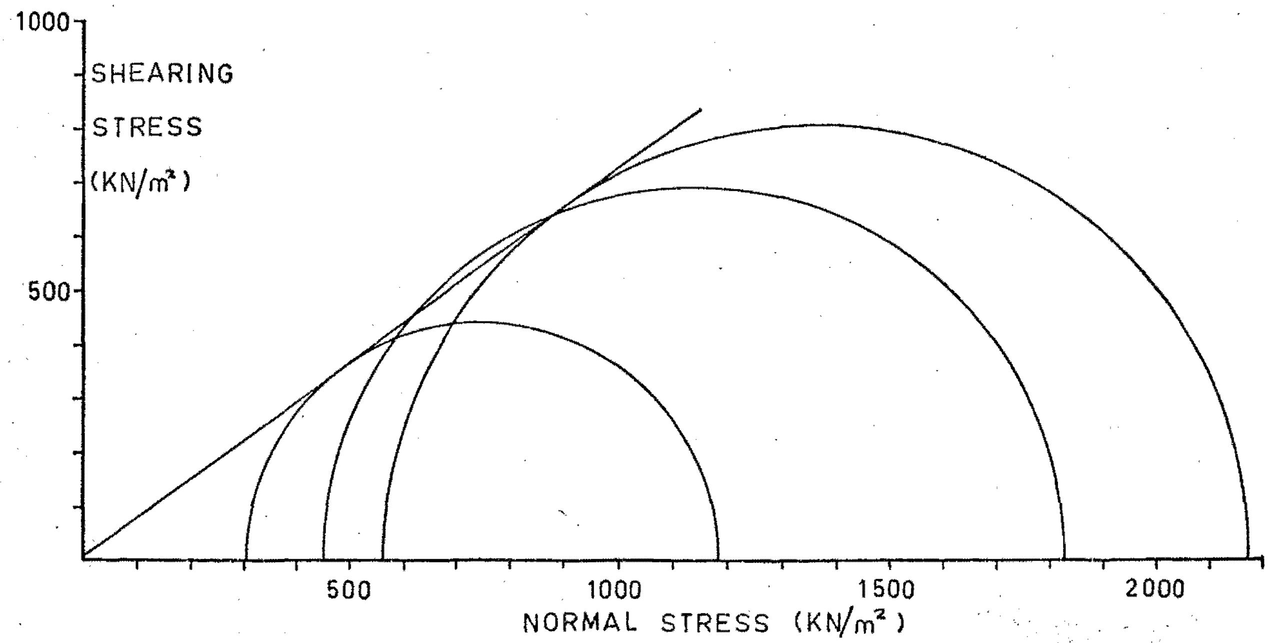


1000
SHEARING
STRESS
(KN/m²)
500
500
1000
1500
2000
NORMAL STRESS (KN/m²)

UNIT 4, BED J



1500
1000
500
DEVIATOR
STRESS
(KN/m²)
CONFINING
PRESSURES
560 KN/m²
450 "
300 "
5
10
15
20
AXIAL LOAD (mm)



1000
500
SHEARING
STRESS
(KN/m²)
500
1000
1500
2000
NORMAL STRESS (KN/m²)

7.4 APPENDIX 4 - MIKONUI EARTHFLOW DRILL CORE LOGS.

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. ONE

PROJECT ... MIKONUI EARTHFLOW . LOCATION .... OARO-KAIKOURA .....

GRID REF .... R.L. GROUND ..... 15.92 .... DATUM ..... MSL. .....

HOLE SIZE 152mm. ANGLE FROM HORIZONTAL 90

GEOLOGICAL INTERPRETATION	graphic log	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample	penetration blows/30cm	relative density	water condition	casing
			Light brown, slighty weathered, silty SAND, with some clay, friable, moist, non plastic.			L		
		14.92						
			Light brownish gray-light rusty brown-dark brown, disturbed, slighty weathered, bentonitic, clayey SILT, with some fine sand, soft, wet, slighty plastic.			L		
		13.92						
						L		
		12.92						
MIKONUI EARTHFLOW FORMATION. A highly variable mixture of light gray-light green -light bluish gray-rusty brown-dark brown, with all structural features (bedding) destroyed, usually slightly weathered, clayey SILT and SAND, with some bentonitic clay, very soft-firm, moist-firm, non plastic-highly plastic and including much fibrous organic material, tree stumps, hard sandstone boulders 2-3m. diam., boulders of silicified wood, and rare iron pyrites nodules.			Light green-light gray-light rusty brown, disturbed, fine SAND, with some silt, friable, wet, non plastic, with organic fibres.			L		
		11.92	Light bluish gray-dark brownish gray-yellowish rusty brown, disturbed, silty BENTONITIC CLAY, soft, wet, moderately plastic, with some angular cobbles and wood fibres.			MD		
		10.92	Light gray, mottled bluish gray green, homogeneous, slightly weathered, sandy SILT, with some clay, soft-hard, moist, non plastic.	piston		MD	CORE AT 10.0	
		9.92					CORE AT 7.2	
			Light greenish gray-dark bluish gray-rusty brown, homogeneous, slightly weathered, silty SAND, with some clay, firm, wet, non plastic, with organic fibres.	spoon		L		
		8.92						
			Light bluish gray-dark reddish brown, slightly weathered, homogeneous, silty SAND, with some clay, firm, moist, non plastic, with some wood fibres.	spoon		L		
		7.92						
			Light brownish gray-light bluish gray, homogeneous, silty fine SAND, firm, moist, non plastic.	spoon		MD		
		6.92						
		5.92						

DRILLER	RELATIVE DENSITY	WATER CONDITION	casing	SAMPLE	ENGNG. GEOLOGY SECTION
Hanwright	VL V. LOOSE	L LOOSING		SPOON Raymond Spoon Sampler	DEPT. OF GEOLOGY
STARTED 8/4/76	L LOOSE	S STATIC		PISTON Stationary Piston Sampling, thinwall and openend tube	UNIV. OF CANTERBURY
FINISHED 6/5/76	MD MED. DENSE	M MAKING			logged ...... G. GROCOTT.
	D DENSE				traced ......"........
	VD V. DENSE				checked ....
					SHEET ....1.... of ....4...

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. ONE

PROJECT ...... MIKONUI EARTHFLOW LOCATION ... OARO-KAIKOURA ......

GRID REF .... R.L. GROUND ...... 15.92 .... DATUM .... MSL. ....

HOLE SIZE ...... 152mm. ANGLE FROM HORIZONTAL ... 90

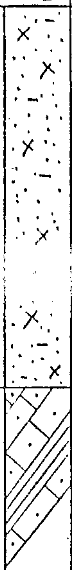
GEOLOGICAL INTERPRETATION	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample	penetration blows/30cm	relative density	water	condition	casing
MIKONUI EARTHFLOW FORMATION	4.92	Light rusty brownish gray, disturbed, clayey SILT, with some sand, soft, wet, slightly plastic, with some rounded, smooth coarse gravels.			MD			
	3.92	Light rusty grayish brown, disturbed, slightly weathered, gravelly COBBLES, irregular, sub angular, rough, with some sand, loose, non plastic.			L			
	2.92	Light greenish gray-light brownish gray, disturbed, silty SAND, firm, wet, non plastic,	spoon		MD			
	1.92							
	0.92							
OKARAHIA SANDSTONE: Light bluish gray-light greenish gray, fissured due to inclusions of dark brown-black, laminated-very thinly layered organic fibres and carbonised wood, fresh, bentonitic CLAY, with some silt, firm-stiff, dry-moist, moderately-highly plastic, grading down wards through light bluish gray gravelly SAND, loose-firm, into light bluish gray, sandy GRAVELS, loose, moist, non plastic.	-.08	Light dull gray, homogeneous, silty SAND, with bentonitic clay, firm, moist, non plastic	spoon		D			
	-1.08	Light greenish gray, homogeneous, clayey silty SAND, firm, moist, slightly plastic.	spoon		D			
	-2.08				D			
	-3.08	Light greenish gray, homogeneous, silty SAND, with some bentonitic clay, firm, moist, slightly plastic, with rare organic particles.	spoon		D	SOAK AT -3.6	▽	
	-4.08							

DRILLER	RELATIVE DENSITY	WATER CONDITION	SAMPLE	
Hanwright	VL V. LOOSE	L LOOSING	SPOON (Raymond Spoon Sampler)	ENGNG. GEOLOGY SECTION
STARTED 8/4/76	L LOOSE	S STATIC	PISTON (Stationary Piston Sampling, thinwall and openend tube)	DEPT. OF GEOLOGY
FINISHED 6/5/76	MD MED. DENSE	M MAKING	casing	UNIV. OF CANTERBURY
	D DENSE			logged ... G GROCOTT ...
	VD V. DENSE			traced ... " ...
				checked ...
				SHEET ... 2 ... of ... 4 ...

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. ONE

PROJECT: MIKONUI EARTHFLOW
LOCATION: OARO-KAIKOURA
GRID REF: ...
R.L. GROUND: 15.92
DATUM: MSL.
HOLE SIZE: 152mm.
ANGLE FROM HORIZONTAL: 90



GEOLOGICAL INTERPRETATION	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample	penetration blows/30cm	relative density	water condition	casing
OKARAHIA SANDSTONE		Light greenish gray, homogen eous, silty SAND, with some bentonitic clay, firm, wet, non plastic, with black organic particles.	▭		D		
	-5.08						
	-6.08	Light greenish gray, fissured, silty SAND, with some benton itic clay, firm, moist, non plastic, with a planar, black, organic inclusion dipping 60° to horizontal.	▭		D		
TORLESSE SUPERGROUP SEDIMENTS Light gray, slightly weathered, fractured and faintly bedded, SANDSTONE and MUDSTONE	-7.08	Light gray, clayey sandy GRAVEL loose, wet, non plastic, with gravels fresh, angular, indurated.			VD		
	-8.08	Light gray, sandy GRAVEL, loose wet, non plastic, with gravels fresh, angular, indurated.					

DRILLER	RELATIVE DENSITY	WATER CONDITION	casing	SAMPLE	ENGNG. GEOLOGY SECTION
Hanwright	VL V. LOOSE			SPOON ▭ Raymond Spoon Sampler	DEPT. OF GEOLOGY
STARTED 8/4/76	L LOOSE	L LOOSING			UNIV. OF CANTERBURY
	MD MED. DENSE	S STATIC		PISTON Stationary Piston Sampling, thinwall and openend tube	logged G. CROCOTT
FINISHED 6/5/76	D DENSE				traced "
	VD V. DENSE	M MAKING			checked
					SHEET 3 of 4

LOG OF DRILL HOLE

HOLE NO. one

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. .... M.W.D. CO-ORD. .... DATUM MSL. H.A.D. GROUND 15.92m.

ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR ....

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour).	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT % 5 10 50	DEPTH H.A.D. Core size casing m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) 50 10 cms 10	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE-DEPTH	R.Q.D.	WATER LEVEL Date	DRILL WATER LOSS % 0-100
TORLESSE SUPERGROUP SEDIMENTS Light gray, fine - medium grained, slightly weathered, SANDSTONE, with occasional very thinly bedded, black MUDSTONE; strong					-7.86 -8.86 -9.86 -10.86 -11.86			Closely spaced joints, with fractures randomly orientated and slightly weathered				

NOTES:

1. Cable Tool from RL. 15.92m. to RL. -7.86m.
2. Rotary from RL. -7.86m. to EOH. at -12.42m.

DRILLER: McVicar

STARTED: ....

FINISHED: ....

DRILL: ....

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG
(cms) 100 50 10 5 1 0.1
1 2 10 20 100 1000
Spacing of natural fractures .... Fractures/m of core

EXPLANATION

LOGGED: G. Grocott

DATE ....

TRACED: ....

CHECKED: ....

VERTICAL SCALE ....

SHEET 4 OF 4

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. TWO

PROJECT ..... MIKONUI . EARTHFLOW. LOCATION ...... OARO-KAIKOURA . . . .

GRID REF. .... R.L. GROUND ...... 23.60m. ...... DATUM ... MSL. ....

HOLE SIZE ...... 152mm. ANGLE FROM HORIZONTAL 90

GEOLOGICAL INTERPRETATION	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	relative density	water condition
			L	
	22.6	Dark grayish brown, disturbed, clayey SILT, very soft, wet, slightly-moderately plastic, highly organic.	L	
	21.6	Light brownish gray, disturbed, silty CLAY, very soft, wet, moderately plastic.		
	20.6	Yellowish brown-greenish gray, disturbed, slightly weathered, clayey sandy SILT, with some medium gravel, very soft, wet, non plastic.	MD	
	19.6	Light bluish greenish gray, homogeneous, SAND, with some silt, soft, moist, non plastic, with a single indurated, fossiliferous, sandstone cobble.	L	
MIKONUI EARTHFLOW FORMATION: A highly variable mixture of light gray-light green-light bluish gray-rusty brown-dark brown, with all original structural features (bedding) destroyed, usually slightly weathered, clayey SILT and SAND, with some bentonitic clay, very soft-firm, moist-wet, non plastic-highly plastic, and including much fibrous organic material, tree stumps, hard sandstone boulders 2-3m. diam., boulders of silicified wood, and rare iron pyrites nodules			L	GONE AT 19.0 ▽
	18.6	Light greenish bluish gray, disturbed, silty BENTONITIC CLAY, with some fine sand and gravel, soft, wet, moderately plastic, slightly organic.		
	17.6	Light gray, disturbed, silty sandy GRAVEL, loose, wet, non plastic, with gravels smooth, sub angular.	MD	
			L	
	16.6	Light grayish brown-black, homogeneous, silty SAND, with some clay, soft-stiff, moist, non plastic.		
	15.6	Greenish brown-dark bluish gray -light grayish blue, homogeneous, silty SAND, with some clay, soft-hard, moist, non plastic.	MD	
	14.6	Light greenish gray, homogeneous slightly weathered, silty SAND, with some clay, soft, moist, non plastic, with soil surfaces tarnishing on exposure.	MD	
	13.6			

DRILLER: Hanwright

STARTED: 17/5/76

FINISHED: 20/5/76

RELATIVE DENSITY	
VL	V. LOOSE
L	LOOSE
MD	MED. DENSE
D	DENSE
VD	V. DENSE

WATER CONDITION: L LOOSING, S STATIC, M MAKING

casing

SAMPLE: SPOON — Raymond Spoon Sampler; PISTON — Stationary Piston Thinwall Openend Tube Sampler.

ENGNG. GEOLOGY SECTION
DEPT. OF GEOLOGY
UNIV. OF CANTERBURY

logged ..... G. CROCOTT ....

traced ....

checked ....

SHEET ... 1 ... of ... 7 ...

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. TWO

PROJECT MIKONUI EARTHFLOW LOCATION OARO-KAIKOURA

GRID REF R.L. GROUND 23.60m. DATUM MSL.

HOLE SIZE 152mm. ANGLE FROM HORIZONTAL 90

GEOLOGICAL INTERPRETATION	graphic log	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample	penetration blows/30cm	relative density	water condition	casing
MIKONUI EARTHFLOW FORMATION		12.6	Light bluish gray, disturbed, sandy coarse GRAVEL, loose, moist, non plastic, with gravels sub angular-angular, smooth.			L	∇	
		11.6						

DRILLER	RELATIVE DENSITY	WATER CONDITION	SAMPLE	
Hanwright			SPOON — Raymond Spoon Sampler	ENGNG. GEOLOGY SECTION
STARTED 17/5/76	VL V. LOOSE			DEPT. OF GEOLOGY
	L LOOSE	L LOOSING	PISTON — Stationary Piston Thinwall Opened Tube Sampler.	UNIV. OF CANTERBURY
	MD MED. DENSE	S STATIC		logged G GROCOTT
FINISHED 20/5/76	D DENSE			traced "
	VD V. DENSE	M MAKING		checked
		casing		SHEET 2 of 7

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 23.6m.

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT % (50 10 5)	DEPTH H.A.D (Core size, casing)	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) (50 10 5 cms)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE DEPTH	RQD	WATER LEVEL (Date)	DRILL WATER LOSS % (0–100)
MIKONUI EARTHFLOW FORMATION								Gray-brown, slightly weathered, SANDSTONE (boulder)				
					11·6, 10·6, 9·6			Light gray-light brown-dark bluish gray-yellowish brown, homogeneous, slightly weathered, silty SAND, with some clay, soft-firm, wet, slightly plastic SM				
					8·6, 7·6, 6·6			Light greenish gray, homogeneous, SAND, with some clay and silt, soft, wet, slightly plastic, with wood fibres and rare fine gravel SM-SC				
					5·6, 4·6, 3·6			Light gray-greenish gray-dark brownish gray, faint organic layering, clayey silty SAND, soft-firm, wet, slightly plastic, partially organic SM-SC				
					2·6			No core				

DRILLER: McVicar

STARTED:

FINISHED:

DRILL:

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG (cms) 100 50 10 5 1 0 — Spacing of natural fractures; 1 2 10 20 100 — Fractures/m of core

EXPLANATION

LOGGED: G. Grocott
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:

SHEET 3 of 7

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 23.6m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (Formation name; Rock or soil type; Description of core)	Depth H.A.D.	Rock structures (or soil description)
MIKONUI EARTHFLOW FORMATION	1.6	No core
	0.6, -0.4, -1.4	Light greenish gray, homogeneous, clayey SAND, with some silt, soft, wet, slightly plastic, with wood fibres and rare sandstone gravels SC
	-2.4	Light gray, slightly weathered, SANDSTONE (boulder)
	-3.4, -4.4	No core
	-5.4, -6.4	Light gray, slightly weathered on fractures, SANDSTONE (boulder)
OKARAHIA SANDSTONE	-7.4	

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: DATE: TRACED: CHECKED: VERTICAL SCALE:

SHEET 4 of 7

HOLE NO. two

LOG OF DRILL HOLE

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 23.6m.

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH H.A.D. (m)	GRAPHIC LOG	FRACTURE LOG	ROCK STRUCTURES (Defects) (OR SOIL DESCRIPTION)	DATE:DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS %
OKARAHIA SANDSTONE. Light bluish gray -light greenish gray, fissured throughout due to inclusions of dark brown-black, laminated-very thinly layered organic fibres and carbonised wood, fresh, bentonitic CLAY, with some silt, firm-stiff, dry-moist, moderately-highly plastic, grading downwards through light bluish gray gravelly SAND, loose-firm, into light bluish gray sandy GRAVELS, loose, moist, non plastic					-8.4 -9.4 -10.4 -11.4			Light bluish gray, homogeneous, bentoitic CLAY, with some silt, firm-stiff, moist, moderately-highly plastic CH - with occassional regular, rounded, polished gravels - highly plastic				
					-12.4 -13.4 -14.4 -15.4 -16.4 -17.4			Light bluish gray-dark brown-light brown-black, very thinly layered-laminated organic and wood fibres, bentonitic CLAY, soft-firm-stiff, moist, highly plastic CH-OH - homogeneous				

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: DATE: TRACED CHECKED: VERTICAL SCALE:

SHEET 5 OF 7

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro
GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 23.6m.
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H A D	ROCK STRUCTURES (Defects) / (OR SOIL DESCRIPTION)
OKARAHIA SANDSTONE	-18.4	- with some black, fine sand lenses
	-19.4	
	-20.4	
	-21.4	Light gray-light greenish gray, homogeneous, bentonitic CLAY, stiff, dry-moist, highly plastic CH
	-22.4	
	-23.4	
	-24.4	- very thinly layered-laminated, black organic inclusions, with rare iron pyrites
	-25.4	
	-26.4	Light grayish green, with black, laminated organic inclusions, bentonitic CLAY, firm-stiff, dry-moist, highly plastic
	-27.4	

Other column headings: WEATHERING (SW MW HW); HARDNESS (H MH MS S); POINT LOAD TEST (kPa); CORE LOSS/LIFT; Core size, casing; GRAPHIC LOG; FRACTURE LOG (Spacing of natural fractures); JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES, FOLIATION, SCHISTOSITY (attitude, thickness, spacing, smoothness) (consistency, compactness, water content, group symbol etc.); DATE/DEPTH; RQD; WATER LEVEL; DRILL WATER LOSS (0-100).

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED
DATE
TRACED
CHECKED
VERTICAL SCALE

SHEET 6 OF 7

LOG OF DRILL HOLE

HOLE NO. two

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro
GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 23.6m.
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D.	ROCK STRUCTURES (Defects) (OR SOIL DESCRIPTION)
OKARAHIA SANDSTONE	-28.4 -29.4	CH-OH - with some sand and silt - grading downwards through a homogeneous silty sandy CLAY into a clayey, silty SAND, soft-loose, non plastic
	-30.4 -31.4 -32.4	Light gray, homogeneous, sandy GRAVEL, loose, moist, non plastic, with rare inclusions of carbonised wood, with gravels regular, rounded, smooth, gap graded GP
	-33.4	

NOTES:
1. Cable Tool from RL. 23.6m. to RL. 12.6m.
2. Rotary from RL. 12.6m. to EOH. at RL.-33.09m.

DRILLER:
STARTED:
FINISHED:
DRILL:

WEATHERING
UW - Unweathered
SW - Slightly weathered
MW - Moderately weathered
HW - Highly weathered
CW - Completely weathered

HARDNESS
VH - Very hard
H - Hard
MH - Moderately hard
MS - Moderately soft
S - Soft
VS - Very soft

FRACTURE LOG (cms)
Spacing of natural fractures
Fractures/m of core

EXPLANATION

LOGGED
DATE
TRACED
CHECKED
VERTICAL SCALE

SHEET 7 OF 7

LOG OF CABLE TOOL DRILL HOLE

HOLE NO. THREE

PROJECT: MIKONUI EARTHFLOW | LOCATION: OARO-KAIKOURA

GRID REF: .... | R.L. GROUND: 41.98m. | DATUM: MSL.

HOLE SIZE: 152mm. | ANGLE FROM HORIZONTAL: 90

GEOLOGICAL INTERPRETATION	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample
MIKONUI EARTHFLOW FORMATION. A highly variable mixture of light gray-light green -light bluish gray-rusty brown-dark brown, with all structural features (bedding) destroyed, usually slightly weathered, clayey SILT and SAND, with some bentonitic clay, very soft-firm, moist-wet, non plastic-highly plastic, and including much fibrous organic material, tree stumps, hard sandstone boulders 2-3m. diam., boulders of silicified wood, and rare iron pyrites nodules.	40.98		
	39.98	Light purplish gray-light rusty brown, disturbed, slightly weathered, silty SAND, with some gravel, very soft, wet, non plastic, with gravels irregular, angular,	
	38.98	Light bluish gray-dark rusty brown, homogeneous, slightly weathered, clayey SILT, with some fine gravel, soft, wet, slightly plastic, with some organic fibres.	Piston
	37.98	Light grayish blue-light to dark yellowish rusty brown, homogeneous, slightly weathered, clayey SILT, with some gravel, soft, moist, moderately plastic, with some organic fibres.	Piston
	36.98		
	35.98	Light grayish yellow, mottled rusty brown, homogeneous, slightly weathered, sandy SILT, with some clay and gravel, firm, moist, non plastic.	Piston
	34.98	Dark gray-dark brown, mottled rusty brown, homogeneous, slightly weathered, silty SAND, with some gravel, firm, moist, non plastic.	Piston
	33.98	Dark rusty brown-light bluish gray, homogeneous, silty CLAY, soft-firm, wet, moderately plastic, with organic fibres	Piston
	32.98	Light greenish gray, disturbed, clayey SILT, soft, wet, slightly plastic, with organic fibres. Light bluish gray-yellowish rusty brown-dark chocolate brown, organic silty CLAY, soft-firm, wet, highly plastic.	Spoon
	31.98		

Relative density (top to bottom): L, L, MD, MD, L, L, L, MD, L, MD, MD, MD, MD, MD, MD, L, L, MD, L

DRILLER	RELATIVE DENSITY	WATER CONDITION	SAMPLE	
Hanwright	VL V. LOOSE	L LOOSING	SPOON — Raymond Spoon Sampler	ENGNG. GEOLOGY SECTION
STARTED 10/5/76	L LOOSE	S STATIC	PISTON — Stationary Piston Thinwalled Openend Tube Sampler.	DEPT. OF GEOLOGY
FINISHED 14/5/76	MD MED. DENSE	M MAKING		UNIV. OF CANTERBURY
	D DENSE	casing		logged G GROCOTT
	VD V. DENSE			traced "
				checked
				SHEET 1 of 5

LOG OF CABLE TOOL DRILL HOLE

HOLE NO.	THREE

PROJECT: MIKONUI EARTHFLOW — LOCATION: OARO-KAIKOURA

GRID REF: — R.L. GROUND: 41.98m. — DATUM: MSL.

HOLE SIZE: 152mm — ANGLE FROM HORIZONTAL: 90

GEOLOGICAL INTERPRETATION	graphic log	metres	RECOVERED DRILL CUTTINGS DESCRIPTION	sample	penetration blows/30cm	relative density	water condition	casing
						L		
MIKONUI EARTHFLOW FORMATION		30.98				L		
			Dark chocolate brown, homogeneous, organic BENTONITIC CLAY, soft-firm, wet, highly plastic.			MD		
						L		
		29.98						

DRILLER: Hanwright	RELATIVE DENSITY	WATER CONDITION	SAMPLE	ENGNG. GEOLOGY SECTION
STARTED: 10/5/76	VL V. LOOSE	L LOOSING	SPOON — Raymond Spoon Sampler	DEPT. OF GEOLOGY
FINISHED: 14/5/76	L LOOSE	S STATIC	PISTON — Stationary Piston Thinwalled Openend Tube Sampler	UNIV. OF CANTERBURY
	MD MED. DENSE	M MAKING		logged: G. GROCOTT
	D DENSE	casing		traced: "
	VD V. DENSE			checked:
				SHEET 2 of 5

LOG OF DRILL HOLE

HOLE NO. three

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 41.98m

ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour))	DEPTH H.A.D. (m)	ROCK STRUCTURES (Defects) / (OR SOIL DESCRIPTION)
Fish-tail bit used between RL. 31.32m. and RL. 17.91m. Samples of washings taken. Driller recorded change in rate of penetration of bit at approx. RL. 22m.	17.91	
OKARAHIA SANDSTONE Light bluish gray-light greenish gray, fissured throughout due to laminated-very thinly layered, dark brown-black inclusions of organic fibres and carbonised wood, fresh, bentonitic CLAY, with some silt firm-stiff, dry-moist, moderately-highly plastic, grading downwards through light bluish gray, gravelly SAND, loose-firm, into light bluish gray sandy GRAVELS, loose, moist, non plastic.	16.91 15.91 14.91	Cream-light gray, occasional very thinly layered carbonised wood inclusions, bentonitic CLAY, with some silt and clay, soft-firm-stiff, moist, moderately plastic, with some tendency towards lensing of various grain sizes CH-OH
	13.91 12.91 11.91 10.91	Light greenish gray-dark chocolate brown, many very thinly layered organic inclusions, bentonitic CLAY, with some silt and sand, firm-stiff, dry-moist, moderately-highly plastic, with many plant remains visible CH-OH

Other column headings: WEATHERING (SW MW HW); HARDNESS (VH H MH MS S); POINT LOAD TEST (kPa); CORE LOSS/LIFT %; Core size, casing; GRAPHIC LOG; FRACTURE LOG (Spacing of natural fractures, cms); DATE/DEPTH; R.Q.D. %; WATER LEVEL (Date); DRILL WATER LOSS % (0–100).

DRILLER: McVicar

STARTED:

FINISHED:

DRILL:

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG (cms) Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: G. Grocott
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:
SHEET 3 OF 5

LOG OF DRILL HOLE

HOLE NO. three

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro
GRID REF. M.W.D. CO-ORD. DATUM MSL. H.A.D. GROUND 41.98
ANGLE FROM HORIZONTAL 90 DIRECTION PHOTO NO. H.A.D. COLLAR

DESCRIPTION OF CORE (FORMATION NAME; ROCK OR SOIL TYPE; DESCRIPTION OF CORE (grain size, texture, mineral content hardness strength, cement & matrix colour))	WEATHERING (SW MW HW)	HARDNESS (H MH MS S)	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD (Core size casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES FOLIATION, SCHISTOSITY (attitude, thickness spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc)	DATE/DEPTH	R.Q.D.	WATER LEVEL	DRILL WATER LOSS %
OKARAHIA SANDSTONE					9.91							
					8.91							
					7.91							
					6.91							
					5.91			Light gray-dark chocolate brown, with very thinly layered organic inclusions, silty, sandy bentonitic CLAY, firm-stiff, dry, moderately plastic, with some very thinly layered clayey sand lenses CL-OH				
					4.91							
					3.91			Light green, homogeneous, bentonitic CLAY, firm-stiff, dry, highly plastic CH				
					2.91							
					1.91			Light gray, homogeneous, bentonitic CLAY, firm, dry, highly plastic - grding downwards into a clayey, silty SAND CH-SC				
					0.91							

DRILLER:

STARTED:

FINISHED:

DRILL:

WEATHERING
UW – Unweathered
SW – Slightly weathered
MW – Moderately weathered
HW – Highly weathered
CW – Completely weathered

HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very soft

FRACTURE LOG (cms)
Spacing of natural fractures 100 50 10 5 1 0
Fractures/m of core 1 2 10 20 100 1000

EXPLANATION

LOGGED:
DATE:
TRACED:
CHECKED:
VERTICAL SCALE:

SHEET 4 OF 5

LOG OF DRILL HOLE

HOLE NO. three

PROJECT MSc Thesis FEATURE Earthflow LOCATION Oaro

GRID REF. .... M.W.D. CO-ORD. .... DATUM MSL. H.A.D. GROUND 41.98

ANGLE FROM HORIZONTAL 90 DIRECTION .... PHOTO NO. .... H.A.D. COLLAR

DESCRIPTION OF CORE FORMATION NAME: ROCK OR SOIL TYPE: DESCRIPTION OF CORE (grain size, texture, mineral content, hardness, strength, cement & matrix colour).	WEATHERING SW MW HW	HARDNESS H MH MS S	POINT LOAD TEST (kPa)	CORE LOSS/LIFT %	DEPTH HAD Core size casing	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures)	ROCK STRUCTURES (Defects) JOINTS VEINS SEAMS SHATTER, SHEAR & CRUSH ZONES FOLIATION SCHISTOSITY (attitude, thickness, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc.)	DATE DEPTH	RQD	WATER LEVEL	DRILL WATER LOSS %
OKARAHIA SANDSTONE					-.09 -1.09 -2.09			Light gray, homogeneous, gravelly SAND, loose, moist, non plastic, with gravels regular, rounded, smooth GP - SAND, with rare gravels SP - sandy GRAVEL, with rare inclusions of carbonised wood				
TORLESSE SUPERGROUP SEDIMENTS Light-dark gray, fine-medium grained, slightly weathered, SANDSTONE, with occasional very thinly bedded, black MUDSTONE, strong					-3.09 -4.09 -5.09			Closely spaced joints, with fractures randomly orientated and slightly weathered				
NOTES: 1. Cable Tool from RL. 41.98m. to RL. 31.32m. 2. Fish-tail bit from RL. 31.32m. to RL. 17.91m. 3. Rotary from RL. 17.91m. to EOH. at RL. -5.87m.					-6.09							

DRILLER: ....

STARTED: ....

FINISHED: ....

DRILL: ....

WEATHERING: UW – Unweathered; SW – Slightly weathered; MW – Moderately weathered; HW – Highly weathered; CW – Completely weathered

HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very soft

FRACTURE LOG (cms): Spacing of natural fractures; Fractures/m of core

EXPLANATION

LOGGED: ....
DATE: ....
TRACED: ....
CHECKED: ....
VERTICAL SCALE: ....

SHEET 5 OF 5